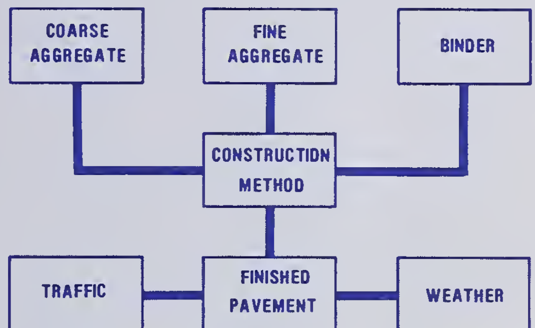
eport No. FHWA-RD-78-209

LTERNATIVES FOR THE OPTIMIZATION OF GGREGATE AND PAVEMENT PROPERTIES ELATED TO FRICTION AND WEAR RESISTANCE



EFFECT	IMPACT
WET SKID RESISTANCE	SAFETY
HYDRDPLANING	SAFETY
TIRE WEAR	$ COST
ROLLING RESISTANCE	$ ENERGY
NDISE LEVEL	PDLLUTIDN
AUTO WEAR AND FATIGUE	$ CDST
FATIGUE & RIDE CDMFORT	SAFETY
GLARE & LIGHT REFLECTION	SAFETY
SPLASH AND SPRAY	SAFETY
APPEARANCE	SAFETY

APRIL 1978
Final Report

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Prepared for
FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
Washington, D. C. 20590

FOREWORD

Under contract with the Federal Highway Administration, an interdisciplinary team of researchers at The Pennsylvania State University has completed a comprehensive review and evaluation of information on properties of aggregates and pavement surfaces related to skid resistance, wear resistance, and other characteristics associated with tire-pavement interactions, such as tire wear, rolling resistance, noise level, and splash and spray.

The study emphasizes the intrinsic properties of aggregates for pavement surfacing systems which may be optimized by the better utilization of existing aggregates and the synthesis of new aggregates. The separate roles of the fine and coarse levels of pavement surface texture are identified for use in estimating skid resistance at any speed. An economic analysis of surfacing systems, using a cost-benefit approach, is also included in the report.

This report is receiving no general distribution.

Charles F. Scheffey

Charles F. Scheffey
Director, Office of Research

TECHNICAL REPORT STANDARD TITLE PAGE

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16. Abstract

Utilizing literature review, expert opinions, research experience and limited tests, the following findings were obtained. Aggregates having a relatively high content of hard minerals accompanied by relatively high levels of differential hardness or porosity and good bonding of grains will resist both polishing and wear. The shape, size and distribution of the aggregate particles and of the grains within the particles also play a significant role in aggregate and surface performance. The levels of friction required by existing traffic and environmental conditions depend upon the presence of adequate surface microtexture at low speeds and the presence of both microtexture and macrotexture at high speeds. Equations to predict skid numbers from texture indicators were developed. Tire noise and tire wear were found to be mainly a function of tire characteristics. However, noise increased as the surface texture became very dense or very coarse and tire wear increased with increasing microtexture or when tire slippage occurred. The effect of aggregate and texture on rolling resistance is small except at extreme macrotexture levels. Open graded surfaces which provide good skid resistance also reduce glare, splash and spray. Known processes and innovations for aggregate beneficiation and for producing synthetic aggregates have been reviewed and discussed. Recommendations are made on properties, processes, testing procedures, and materials that appear promising and warrant further studies. An economic evaluation of various surfacing systems revealed that the costs of tire wear, accidents and noise are significant in that order, and that a "Systems Cost" is the most important consideration in choosing the system.

17. Key Words Aggregates, Microtexture, Macrotexture Skid Resistance, Wear Resistance, Noise, Tire Wear, Rolling Resistance, Glare, Splash, Spray, Economic Analysis		18. Distribution Statement No restrictions. This document is available through National Technical Information Service, Springfield, Virginia 22161.	
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Several individuals, agencies and firms have contributed to information collection and constructive suggestions, including personnel from the Pennsylvania Department of Transportation, Pennsylvania Aggregates Association, GE Refractories Company, Carborundum Company, Pittsburgh-Corning Corporation, the Glen-Gery Corporation, Hydraulic Press Brick Company, Republic Steel, Lone Star Industries, Inc., Hudson Valley Solite, National Ash Association, Expanded Shale, Clay and Slate Association, and others whose interest and expert opinions are appreciated.

The contents of this report reflect the views of the principal investigators and the project staff, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policy of the Department of Transporation. This report does not constitute a standard, specification, or regulation.

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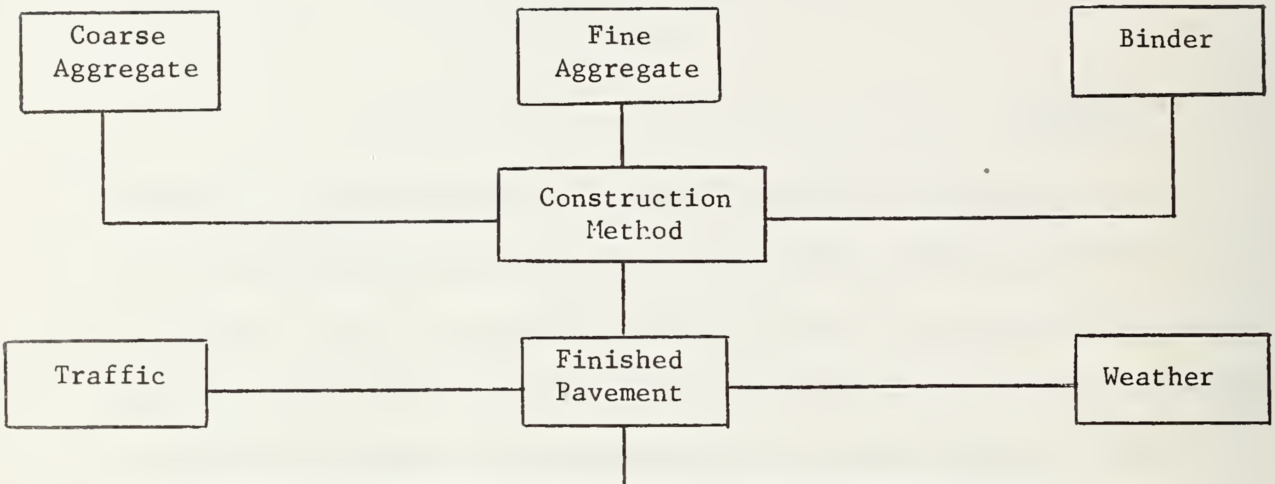
1.0 INTRODUCTION

1.1 Background

For the purposes of skid resistance and wear resistance, a pavement surface system is only as good as the materials it is made from and the method for incorporating these materials to form the system. Normally, aggregates constitute 70 to 95 percent by weight of the surface material. The research reported here is concerned with (1) the identification and evaluation of existing and proposed skid-resistant and wear-resistant aggregate and pavement surface systems, and (2) the development of recommendations for material systems that will meet surface performance requirements of skid resistance and wear resistance while taking into consideration other performance properties including tire wear, noise level, rolling resistance, glare and light reflection, and other functional surface characteristics associated with traffic, geometric, and environmental conditions. The role of aggregates on surface properties and the subsequent penalties and benefits are shown in Figure 1.1. Broad relationships between surface materials, design and construction, and surface performance are summarized in the block diagram, Figure 1.2.

1.2 Objectives

The specific objectives of this research are:



Effect	Impact
Wet Skid Resistance	Safety
Hydroplaning	Safety
Tire Wear	$ Cost
Rolling Resistance	$ Energy
Fuel Economy	$ Energy
Noise Level	Pollution
Auto Wear & Fatigue	$ Cost
Fatigue & Ride Comfort	Safety
Glare & Light Reflection	Safety
Appearance (color, etc.)	Safety
Splash and Spray	Safety

Figure 1.1. Role of Aggregates on Surface Properties and the Subsequent Penalties and Benefits of the Surface Properties

AGGREGATE PROPERTIES

Mineralogy

Percent mineral composition, mineral hardness, crystallinity, grain shape, size, and distribution, grain bonding, chemical stability of minerals

Other Properties

Particle shape, size, gradation, strength, toughness, resilience, abrasion, polishability, porosity, absorption, density, etc.

BINDER PROPERTIES

Bituminous Materials

Viscosity, ductility, temperature sensitivity, weathering susceptibility

Portland Cement Paste

Quality of cement (fineness, strength, impurities), quality of mixing water (potable)
Admixtures

PAVEMENT SURFACE DESIGN

Bituminous Mixtures

Aggregate gradation, asphalt grade and content in mix, mixing, placing, compaction, surface texture, permeability

P.C.C.

Strength (cement factor, water to cement ratio, mixing, placing, curing), durability (air-entrainment), elasticity, alkali reactivity, quality of surface aggregate (particularly sand), texturing

SURFACE PERFORMANCE

Wear and Skid Resistance

a. Wear on macro-scale will cause loss of macrotexture. May cause surface deterioration due to abrasion, degradation, pitting, raveling, rutting, weathering.

b. Wear on a micro-scale may cause loss of safe skid resistance due to polishability of exposed aggregates and loss of surface particles due to poor bonding (loss of microtexture).

Other

Tire wear, noise generation, light reflection, splash and spray, etc.

Figure 1.2. Relationships between Surface Materials and/or Design and Surface Performance

1. To determine optimal aggregate properties and pavement surface characteristics needed to provide and retain desirable functional performance requirements and to establish warrants for the use of optimal pavement surfacing systems.
2. To explore the range of candidate aggregates and pavement surfacing strategies to meet requirements.
3. To conduct economic analyses and to make recommendations and plans for future action.

1.3 Scope of Work

This study primarily involves the use of existing sources of information, published and unpublished, and consultations with experts in the several technologies involved. The emphasis in this study is on the intrinsic properties of aggregates for pavement surfacing systems to optimize the modification of existing aggregates and the synthesis of new aggregates.

1.4 Approach

Six tasks and several subtasks were delineated as guidelines in meeting the objectives of the research. The six tasks are:

Task A -- Quantification of Optimal Properties of Pavement Surfaces and Aggregates

Task B -- Criteria for Use of Optimal Pavement Surfaces

Task C -- Aggregate Selection and Evaluation

Task D -- Technical Evaluation of Pavement Surfacing Systems

Task E -- Economic Evaluation of Alternative Aggregates and Pavement Surfacing Systems

Task F -- Synthesis of Results and Recommendations

2.0 OPTIMAL PAVEMENT SURFACE PROPERTIES

2.1 The Relationship between Skid Resistance and Pavement Texture

When water is present on the road surface, it forms a lubricating film between the tire and the pavement and thus reduces the direct contact between them. Therefore, wet pavement friction is less than dry pavement friction. In a survey of motor vehicle accidents on California state highways in 1970, Satterthwaite [1976] found that the frequency of accidents tended to increase in wet weather. The ratios of accident rates on wet days and on dry days were 1.85 and 1.80 for single-vehicle accidents and head-on collisons, respectively.

As the vehicle speed increases, it becomes more difficult for water to escape from under the tire. This water acts as a lubricant and the skid resistance decreases. The rate of decrease of skid resistance with speed varies for different surfaces. Some pavements may have high skid resistance at low speeds, but relatively low skid resistance at high speeds. On the other hand, some pavements have intermediate skid resistance at low speeds but maintain adequate resistance as speed increases.

The measure of skid resistance is the skid number (SN), defined as 100 times the coefficient of friction measured in accordance with ASTM Method of Test E 274-77 [1977].

The variation of skid number with speed is specified by the skid number gradient (SNG) or the percent skid number gradient (PSNG). The former is defined as the rate of decrease in skid number with increasing speed, while the latter is defined as the percentage of the rate of decrease in skid number with increasing speed.

Skid number is a function of road surface geometry, i.e., pavement texture. Pavement texture is subdivided into macrotexture and microtexture. Macrotexture, which is on the scale of the aggregate gradation, provides passages for water to escape from the tire-pavement interface. Microtexture, which is the small-scaled texture on the scale of the surface roughness of aggregate particles, penetrates waterfilm to provide direct contact with the tire. A surface which has good microtexture ensures a high skid number at low speeds. Macrotexture becomes more important as speed increases. The combined effects of macrotexture and microtexture were reported by Kummer and Meyer [1967]. In summary they concluded that at low speeds skid number is determined by microtexture, while at high speeds both microtexture and macrotexture are important.

2.2 Significance of Texture Study

From a safety point of view, there seems to be no maximum requirement; the skid number should be as high as possible. However, when other effects such as cost, noise, tire wear, etc., are considered, there may be an optimum skid resistance. A quantitative study of the effects of pavement texture on skid resistance and these other effects is necessary to determine the optimum surface. Some of the points that should be considered are:

(1) Some data show that the frequency of wet pavement accidents decreases sharply as skid number (SN_{40}) increases from a small value, but remains about the same after skid number becomes higher than 45 [Mahone and Runkle 1972, Rizenbergs et al. 1974]. The existence of a break point above which skidding accidents cannot be reduced significantly appears to be a reasonable assumption. However, the level of this critical skid

resistance value and the speed at which it is measured would be expected to depend on local conditions and traffic demands.

(2) The costs of producing and maintaining a high level of skid resistance.

(3) The noise produced by pavements, reported to be lower on coarser surfaces by some researchers [Corcoran 1972, Veres 1974] and higher by others [Gordon et al. 1971], appears to be high on both very smooth and very coarse surfaces.

(4) Tire wear increases as microtexture increases [Bond 1974] and may be affected by macrotexture, although there is insufficient data in the literature to establish the nature of this effect. The general trends have been hypothesized by Moore [1975].

(5) Skid resistance requirements differ with the traffic demands for speed, cornering, and braking.

(6) Skid resistance varies significantly with weather conditions, often increasing dramatically after rainfall [Rice 1977, Dry et al. 1977] and decreasing during dry periods. Texture measurements are not able to account for any seasonal variations due to pavement contamination (dust particles, oil films, etc.). Therefore, any relationship between texture and skid resistance must be limited to a comparison of skid resistance measurements made subject to identical climate and weather conditions.

2.3 A Model for the Skid Number-Speed Relationship

In order to establish the relationship between pavement texture and the skid number at any speed, it is necessary first to establish a model for the skid number behavior with speed. Three models were considered, and one was chosen for this study on the basis of its suitability for subsequent correlation with texture data.

The most commonly used relationship to fit skid number (SN)-speed (V) data is Meyer et al. [1974]:

$$SN = a_o + a_1 V + a_2 V^2 . \qquad \text{2-1}$$

In this model, a_o is representative of the low-speed skid resistance and might be expected to correlate with some measure of microtexture. The skid number gradient (SNG) for this model is:

$$SNG = - \frac{d\,(SN)}{dV} = - (a_1 + 2\, a_2\, V) \qquad \text{2-1a}$$

and the percent skid number gradient (PSNG) is:

$$PSNG = \frac{SNG}{SN} \times 100 = \frac{-(a_1 + 2\, a_2 V)\, 100}{a_o + a_1 V + a_2 V^2} . \qquad \text{2-1b}$$

It has been reported that macrotexture parameters and PSNG are highly correlated [Henry et al. 1975, Veres et al. 1975] but Equation 2-1b contains a parameter (a_o) which is highly dependent on microtexture. For this reason and because of the complex nature of Equation 2-1b, this equation was rejected.

Majcherczyk [1974] plotted skid number-speed data on log-log paper and obtained a linear fit, implying a model expressed as:

$$SN = b_o\, V^{b_1} . \qquad \text{2-2}$$

The gradients for this model are:

$$SNG = - b_o\, b_1\, V^{(b_1 - 1)} \qquad \text{2-2a}$$

$$PSNG = - \frac{b_1}{V}\, 100 . \qquad \text{2-2b}$$

Majcherczyk found a correlation between b_1 and macrotexture which is consistent with earlier findings that PSNG at a given speed is a function of macrotexture alone. The deficiency of the model lies in the behavior of skid number at low speed. Since SN decreases with speed, b_1 must be negative, with the result that the model predicts skid numbers that are too high at low speeds. Therefore, the parameter b_o cannot be correlated with microtexture parameters.

A third model was considered for this study:

$$SN = c_o \, e^{-c_1 V} . \qquad \text{2-3}$$

Before discussing the advantages of this model, it is necessary to test its ability to fit actual data over a wide range of test speeds. SN data collected during the 1976 test season on six pavements at test speeds from 10 to 50 mph (17 to 80 km/h) was used to test the model Equation 2-3. Least squares regression analyses were performed producing the results shown in Figure 2.1 It can be seen that the model fits the data extremely well, within the precision of the test procedure. Data over a more limited speed range of 30 to 60 mph (50 to 100 km/h) were also available and were used to test the validity of the model. In all cases, the model Equation 2-3 provided an excellent fit to the data. This form of relationship was also used to correlate outflow data in England [Lees et al. 1974].

As in the first model, low-speed skid resistance is indicated by the zero-speed intercept of the curve fitting the data. In this case c_o is the zero-speed intercept and subsequently will be shown to be strongly correlated with microtexture parameters. The gradients for the model Equation 2-3 are:

$$SNG = c_o \, c_1 \, e^{-c_1 V} \qquad \text{2-3a}$$

$$PSNG = c_1 \, (100). \qquad \text{2-3b}$$

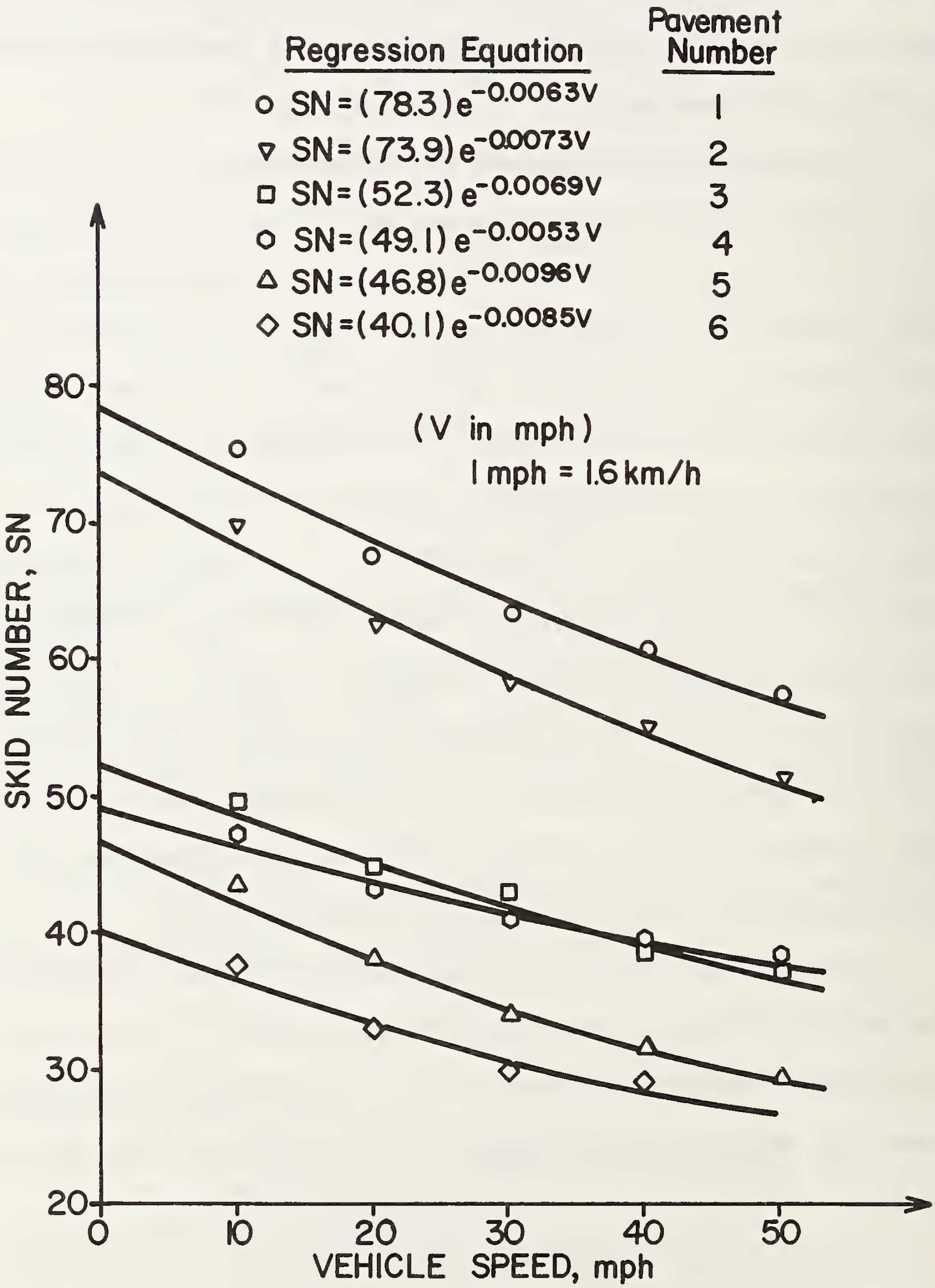


Figure 2.1. Skid Number vs. Vehicle Speed

For this model, the parameter c_1 is proportional to PSNG and therefore should be a function of macrotexture parameters only. This will be verified later. Note from Equation 2-3b that the PSNG is independent of speed. This is consistent with research [ENSCO 1977] which indicated that the PSNG derived from data obtained for a wide variety of surfaces varied only slightly with speed. Further, this slight dependence was sometimes an increase and sometimes a decrease with speed. It was concluded, therefore, that PSNG could be considered to be a very weak function of speed over the range tested. Thus, from the definition of PSNG:

$$PSNG \equiv \frac{-100}{SN} \frac{d\ (SN)}{dv} .$$

Rearranging and integrating from V = o to V:

$$\int_{SN_o}^{SN_V} \frac{d(SN)}{SN} = -\frac{PSNG}{100} \int_o^V dv$$

produces the proposed Penn State Model for skid resistance-speed behavior:

$$SN = SN_o\ e^{-\left(\frac{PSNG}{100}\right)V} \qquad 2\text{-}4$$

where SN_o = the zero-speed intercept, a function of microtexture

PSNG = the percent-skid-number-gradient, a function of macrotexture

V = mph (1 mph = 1.61 kmph)

The significance of this finding is that it will decouple the effects of microtexture and macrotexture if it is possible to obtain microtexture parameters which predict SN_o and macrotexture parameters which predict PSNG.

Findings in earlier research appear to be consistent with the Penn State Model. Sabey [1966] and Gallaway and Rose [1970] found a higher correlation between macrotexture and PSNG than between macrotexture and SNG. Schulze [1968] using model surfaces concluded that a surface with high microtexture produces high skid numbers at low speeds and a steeper gradient (SNG) than low microtexture surfaces. Mahone [1975] found that the surfaces of actual pavements which originally had steep gradients would have intermediate gradients after polishing (wearing away of microtexture).

2.4 Measurement and Specifications of Pavement Texture

It has been customary to subdivide pavement texture into microtexture and macrotexture. The division between the two, however, has not yet been determined rationally. A detailed power level of the entire pavement spatial frequency spectrum, from fine microtexture to coarse macrotexture, provides for a much finer subdivision. Power spectrum analysis [Lawther et al. 1974] can produce 20 or more third-octave subdivisions of pavement texture, each subdivision providing a texture parameter. Detailed spectral analysis is useful for research purposes and will be used subsequently to justify the selection of a division between macrotexture and microtexture.

Since the purpose of this study is to aid in the evaluation of aggregates for paving systems, the emphasis is on techniques which can be applied to small samples and which provide information for selecting the geometrical configuration of aggregates. For this purpose, it is desirable to avoid full-scale tests which require actual highway construction or which measure texture indirectly as, for example, the use of tire noise signature to predict macrotexture [Veres et al. 1975]. Profiling techniques and the

British Portable Tester can be utilized on laboratory samples before and after polishing or abrasive wear. Visual observations, made either by direct examination of the surface or by stereophotography [Schonfeld 1974], can also be used on small samples, but this method has not been proven to yield reliable results.

2.4.1 Microtexture Parameters

Profiling techniques and the British Portable Tester appear to be most promising for evaluating the microtexture of potential aggregates. The advantage of the British Portable Tester [Giles et al. 1964, ASTM 1976] is that it provides a rapid microtexture evaluation without the need for complex data reduction procedures. Profiling techniques [Lawther et al. 1974] provide more information on the actual nature of the aggregate and its microtexture. Microtexture profile data can be processed in a variety of ways to obtain:

1. Root Mean Square Texture Height (RMSH) $\equiv [\frac{1}{L} \int_o^L (y-\bar{y})^2 \, dx]^{1/2}$
2. Arithmetic Mean Texture Height (AMH) $\equiv \frac{1}{L} \int_o^L |y-\bar{y}| \, dx$
3. Root Mean Square Texture Slope (RMSS) $\equiv [\frac{1}{L} \int_o^L (\frac{dy}{dx})^2 \, dx]^{1/2}$

where y is the local height above an arbitrary reference plan and x is the distance along the profile of length L. These parameters may be obtained from the direct processing of microtexture profiles or they may be derived from the power spectrum of the profile. The values of the parameters depend on the cut-off frequency (f_c) which separates the microtexture and macrotexture ranges.

2.4.2 Macrotexture Parameters

For the evaluation of the macrotexture of potential pavement systems, profiling can be used to provide data relating to the geometry of the surface.

Profiles of candidate systems can be processed to yield a variety of pavement macrotexture parameters:

1. Root Mean Square Texture Height (RMSH)
2. Arithmetic Mean Texture Height (AMH)
3. Root Mean Square Texture Slope (RMSS)
4. Mean Hydraulic Radius (MHR) [Moore 1966]
5. Profile Ratio [Sabey 1966]
6. Mean Texture Depth (MD) [ACPA 1969].

As in the case of the microtexture parameters, these parameters may be derived from profiles using direct or spectral analysis techniques. However, the RMSH and AMH are insensitive to the cutoff frequency separating microtexture and macrotexture since the higher space frequencies do not contribute significantly to the values of these parameters.

2.5 Correlation of Texture Parameters and Skid Number Parameters

It is desirable to use the least number of texture parameters which provide the necessary information to predict pavement performance. The two parameters selected for further evaluations, using the data of the 1976 test season (Figure 2.1 and Table 2.1), were the root mean square texture heights of the microtexture and the macrotexture. In addition, the British Portable Number (BPN) was considered as an alternative microtexture parameter.

2.5.1 Correlation of SN_o with RMSH Microtexture

As mentioned previously, the value of RMSH of the microtexture depends on the lowest space frequency (f_c) which is considered in the microtexture. Table 2.2 presents the RMSH for cutoff frequencies from 400 to 16,300 cycles per meter. These were computed from the power spectrum level data for the six surfaces using the relationship:

Table 2.1. Skid Numbers Measured at 10, 20, 30, 40, and 50 mph (16, 32, 48, 64 and 80 km/h) and PSNG Obtained from Least Squares Fit to Equation 2.4

Pavement No.	SN_{10}	SN_{20}	SN_{30}	SN_{40}	SN_{50}	SN_o	PSNG* (hr/mi)
1	75.3	67.7	63.6	61.2	57.8	78.3	0.63
2	70.0	62.7	58.5	55.5	51.6	73.9	0.73
3	49.4	44.7	43.0	39.0	37.4	52.3	0.69
4	47.9	43.4	40.9	39.5	38.6	49.1	0.53
5	43.7	38.0	34.2	31.8	29.6	46.8	0.96
6	37.9	33.0	30.0	29.5	-	40.1	0.85

* 1 hr/mi = .625 hr/km

Table 2.2 Microtexture Values

Pavement No.	1	2	3	4	5	6
BPN	88.2	75.7	69.4	66.6	61.8	56.3
Cutoff Frequency (cycles/meter)	Root Mean Square Texture Height (microinches) (1 microinch = .025 micron)					
400	1199	1168	1263	665	951	1091
500	1102	1083	1117	599	854	883
630	937	936	963	559	778	796
800	820	780	848	517	698	684
1000	726	696	694	473	612	597
1250	628	671	615	446	529	490
1630	549	584	528	419	440	416
2000	460	497	450	401	373	349
2500	377	436	367	359	314	294
3150	307	347	295	319	258	225
4000	253	286	234	290	207	180
5000	199	225	188	270	165	144
6300	162	178	152	242	133	116
8000	128	144	121	218	107	93
10,000	98	122	97	198	84	74
12,500	76	103	80	175	66	60
16,300	59	85	66	155	51	49

$$\mathrm{RMSH}\ (f_c) = \sqrt{\int_{f_c}^{\infty} S(f)\ df} \simeq \sqrt{\sum_{i=c}^{\infty} S(f_i)\ \Delta f_i} \qquad 2\text{-}5$$

where $S(f)$ = mean square spectral density;

f_i = center frequency, ith third octave band; and

Δf_i = frequency interval of ith third octave band.

A linear regression of RMSH at each cutoff frequency with the values of SN_o from Table 2.1 was performed. The correlation coefficient for each cutoff frequency was determined and is shown in Figure 2.2. The cutoff frequency with the highest correlation coefficient is 1630 cycles per meter, with 2000 cycles per meter (0.5 mm wavelength) providing only slightly lower correlation. Based on this evidence, for skid resistance purposes microtexture should be defined as consisting of asperities that are less than 0.5 mm wide. This is consistent with earlier observations where the 0.5 mm width has been suggested [Rice 1977].

A least squares regression analysis produces the following relationship:

$$SN_o = -44.4 + 240\ (\mathrm{RMSH})_{2000} \qquad 2\text{-}6$$

where RMSH is the root mean square height of the microtexture in milliinches (1 milliinch = 25.4 microns) with a cutoff frequency of 2000 cycles per meter.

2.5.2 Correlation of SN_o with BPN

Averages of British Portable Numbers measured on the test sites are given in Table 2.2. An extremely good correlation was observed between BPN and SN_o as shown in Figure 2.3. The correlation equation is:

$$SN_o = -34.9 + 1.315\ (\mathrm{BPN}). \qquad 2\text{-}7$$

An analysis of 20 sites in West Virginia [Leu 1977] provided a similar result:

$$SN_o = -31 + 1.38\ \mathrm{BPN}. \qquad 2\text{-}7a$$

Cutoff Frequency x 100	4	5	6.3	8	10	12.5	16.3	20	25	31.5	40	50	63	80	100	125	163
Correlation Coefficient	.440	.607	.559	.530	.666	.797	.875	.872	.820	.731	.602	.346	.221	.118	.055	.000	.032

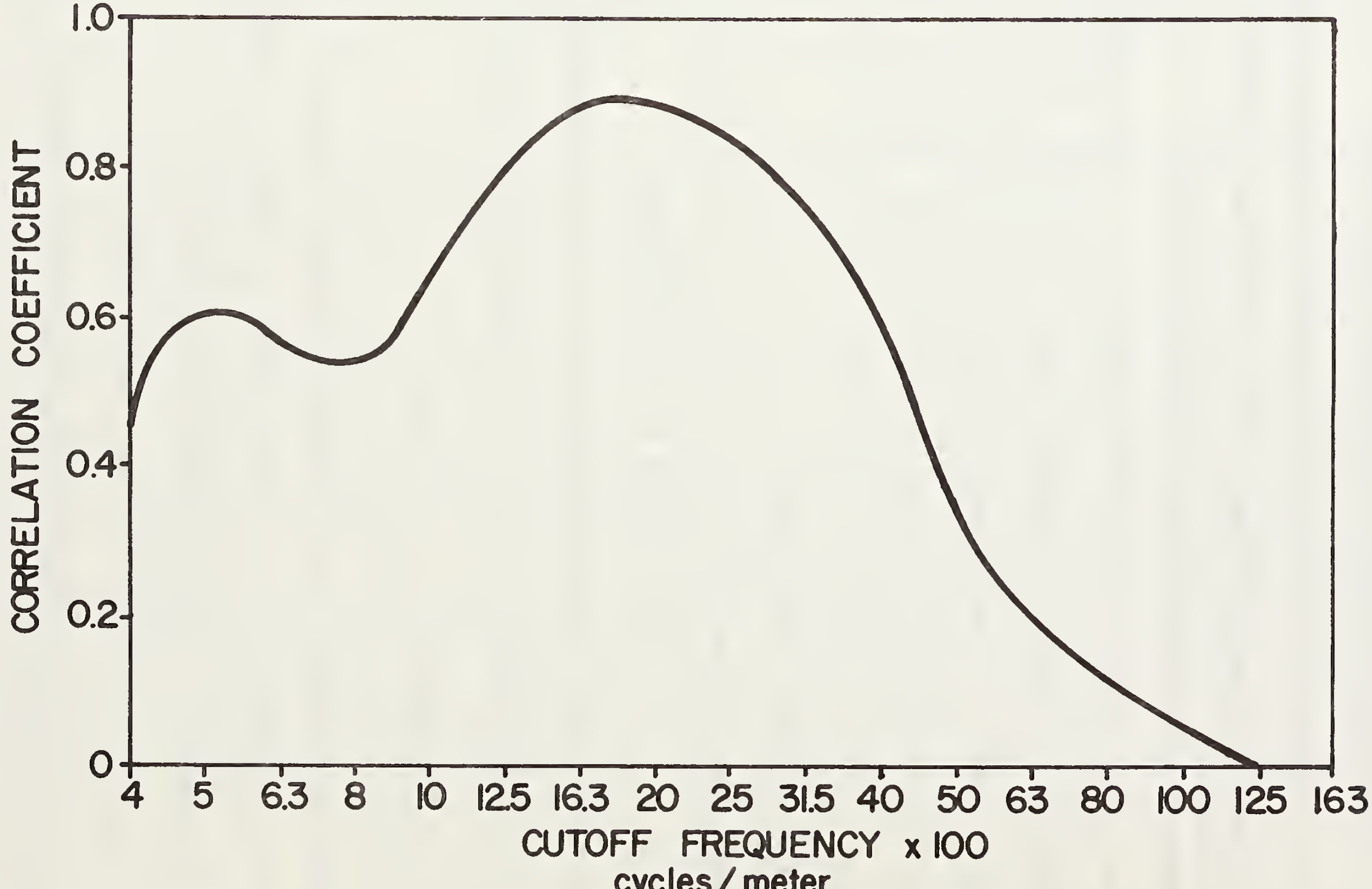


Figure 2.2. The Correlation Coefficient between Zero-Intercept Skid Number and Root Mean Square Texture Height at Different Cutoff Frequencies

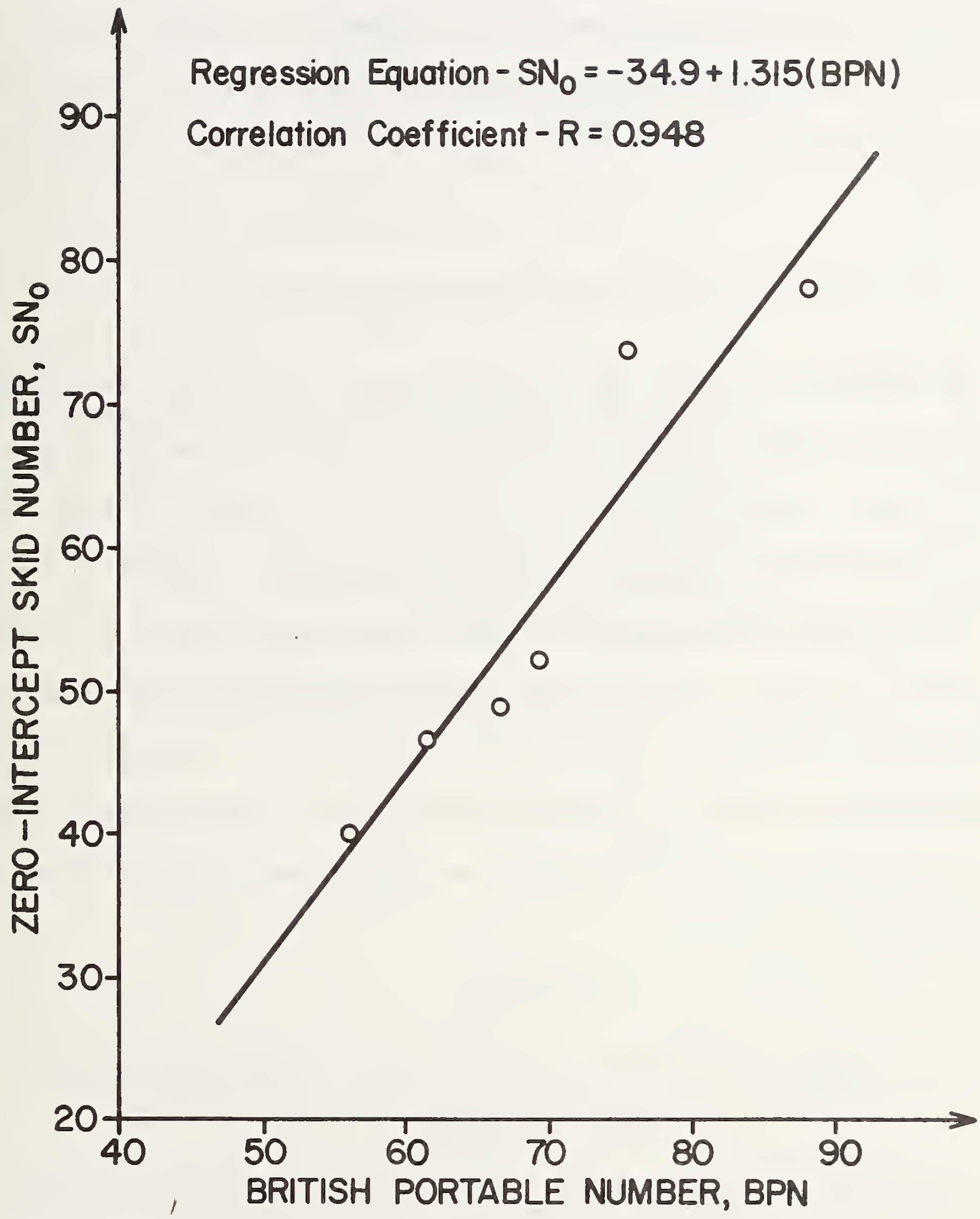


Figure 2.3. Zero-Intercept Skid Number vs. British Portable Number

Two reasons for the superiority of the correlation of BPN with SN_o may be that BPN is a direct measurement of friction at about 7 mph (11 km/h) and that it is sensitive only to the microtexture of the top surfaces of the aggregate. The microtexture profiles include the microtexture of the valleys, which do not play an important role in providing skid resistance. The valleys are often different from the peaks, particularly on polished surfaces. However, the profiling technique does provide quantitative information on the size of the microtexture which is useful information in the design of aggregates.

2.5.3 Correlations of PSNG and RMSH Macrotexture

Macrotexture data from the six test sites in the study are tabulated in Table 2.3. RMSH and AMH were obtained by direct processing of the profile data. Sandpatch data for mean texture depth (MD) are also included. The strong correlation between these three parameters is shown in Figure 2.4, which suggests that any one could be used as a macrotexture parameter with similar results. However, the sandpatch method is deemed less suitable than profiling techniques for laboratory evaluation. Furthermore, RMSH macrotexture was chosen over AMH arbitrarily as the macrotexture parameter. The correlation between RMSH and PSNG was investigated using a relationship of the form:

$$y = d_o \, x^{d_i}$$

with a least square regression resulting in the recommended equation:

$$PSNG = 1.50 \; RMSH^{-.28}. \qquad 2\text{-}8$$

When data from the parallel study on the 20 pavements in West Virginia [Leu 1977] are used, a result indicating a higher sensitivity to macrotexture is obtained:

$$PSNG = 3.8 \; RMSH^{-.52} \qquad 2\text{-}8a$$

Table 2.3 Macrotexture Parameters

Pavement No.	RMSH (milli-inches)[1]	AMH (milli-inches)[1]	MD (milli-inches)[1]
1	28.1	21.3	62
2	8.24	6.28	27
3	21.6	16.6	55
4	20.7	15.9	53
5	6.27	4.67	18
6	11.3	8.41	20

NOTES: RMSH = Root Mean Square Texture Height

AMH = Arithmetic Mean Texture Height

MD = Mean Texture Depth (Sandpatch)

1. 1 milliinch = 25.4 microns

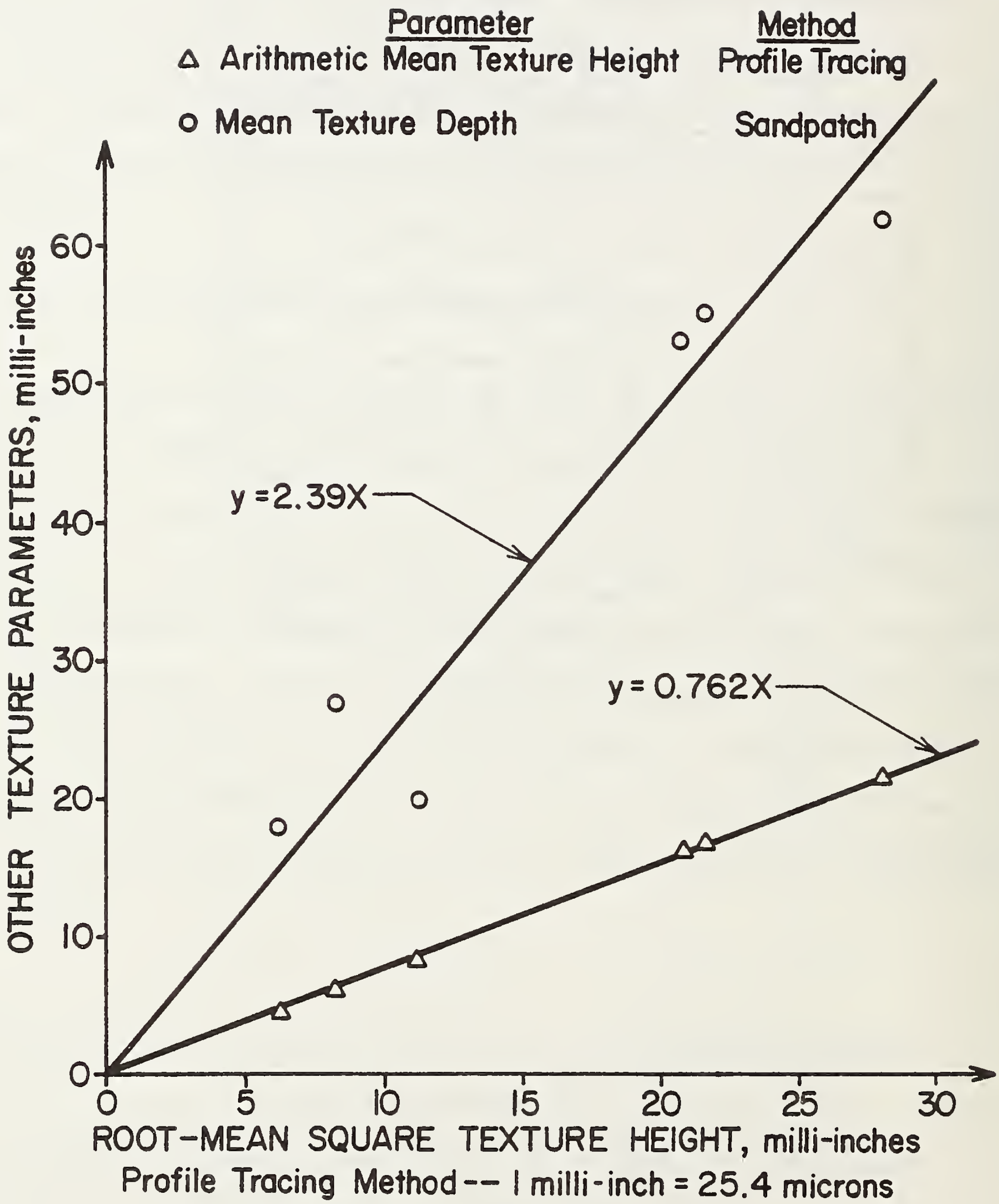


Figure 2.4. The Relationships of Macrotexture Values

Since this result (Equation 2-8a) is based on a larger number of pavements over a wider range of texture, it is recommended that this equation be used until the data base is further increased.

2.5.4 Prediction of Skid Resistance from Texture Parameters

By combining Equations 2-4 with Equations 2-6 and 2-8, skid numbers (SN) can be predicted from the root mean square height of the microtexture and macrotexture ($RMSH_{2000}$ and RMSH):

$$SN = (240\ RMSH_{2000} - 44.4)\ e^{-.015V}\ (RMSH)^{-.28} \quad . \qquad 2\text{-}9a$$

An alternative prediction can be obtained by combining Equation 2-4 with Equations 2-7 and 2-8a, thereby relating skid number with BPN and RMSH macrotexture:

$$SN = (1.315\ BPN - 34.9)\ e^{-.015V}\ (RMSH)^{-.28} \qquad 2\text{-}9b$$

Table 2.4 presents the measured skid numbers and the skid numbers as predicted by the two methods. The results are plotted in Figure 2.5, which shows good agreement between predicted and measured skid numbers.

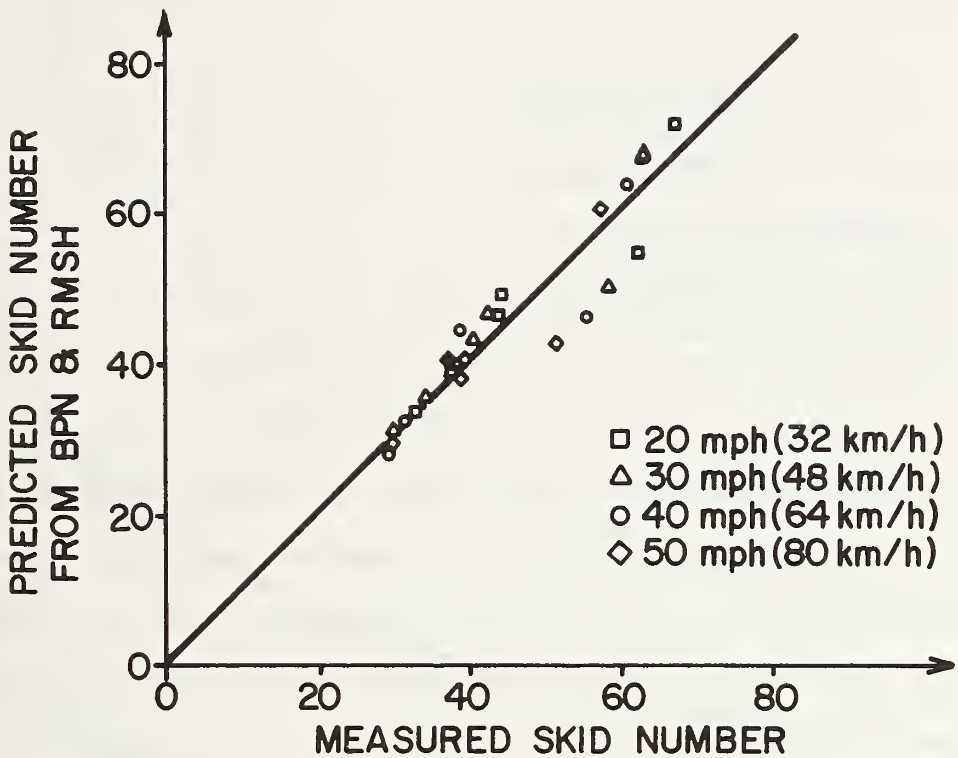
The larger data sample obtained in West Virginia [Leu 1977] is subject to different weather conditions and, therefore, cannot be directly compared with the above results. Those results provide the following relationship between skid resistance and the texture parameters:

$$SN = (1.38\ BPN - 31)\ e^{-.038V}\ RMSH^{-.52} \qquad 2\text{-}9c$$

Finally, when only sandpatch data (MD) are available for macrotexture measurements, the correlation shown in Figure 2.4 can be combined with Equation 2-9c to estimate skid resistance.

Table 2.4. Measured and Predicted Skid Numbers

Pavement No.	1	2	3	4	5	6
Speed mph (km/h)	Measured Skid Number					
20 (32)	67.7	62.7	44.7	43.4	38.0	33.0
30 (48)	63.6	58.5	43.0	40.9	34.2	30.0
40 (64)	61.2	55.5	39.0	39.5	31.8	29.5
50 (80)	57.8	51.6	37.4	38.6	29.6	
Speed mph (km/h)	Predicted Skid Number from BPN and RMSH Equation (2-9b)					
20 (32)	72.1	54.7	49.7	46.4	38.8	33.6
30 (48)	68.0	50.4	46.6	43.5	35.5	31.1
40 (64)	64.1	46.3	43.8	40.8	32.4	28.9
50 (80)	60.4	42.7	41.1	38.3	29.7	26.7
Speed mph (km/h)	Predicted Skid Number from $(RMSH)_{2000}$ and RMSH Equation (2-9a)					
20 (32)	58.7	63.4	56.0	45.6	37.7	33.8
30 (48)	55.3	58.3	52.6	42.8	34.5	31.3
40 (64)	52.1	53.7	49.3	40.0	31.5	29.0
50 (80)	49.2	49.4	46.3	39.6	28.8	26.9



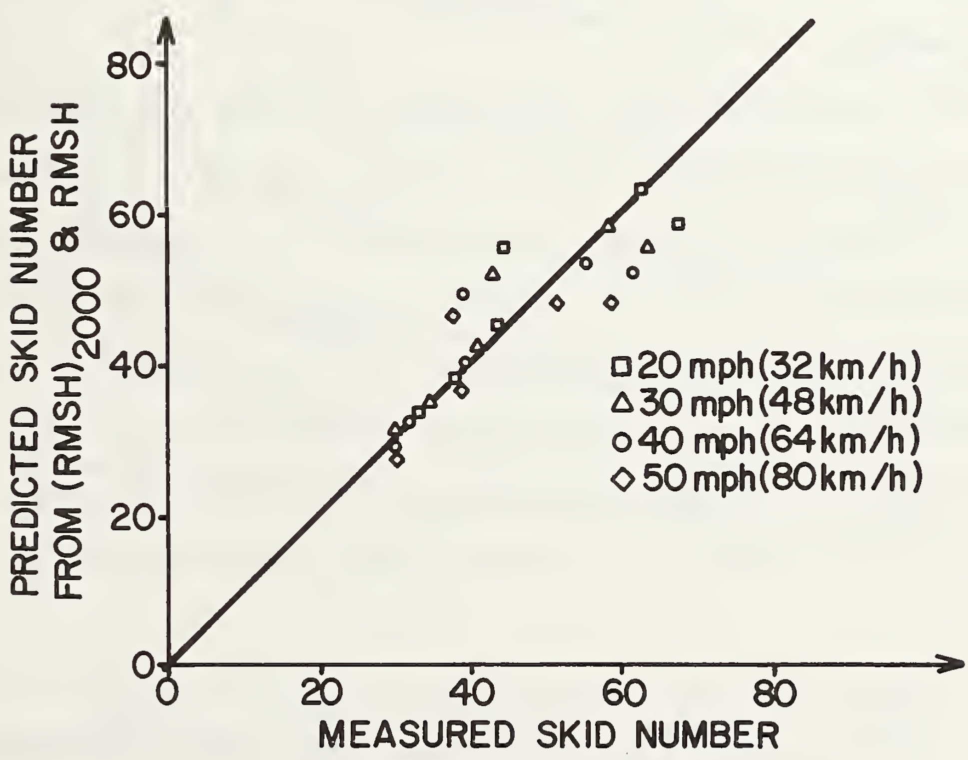


Figure 2.5. Predicted Skid Number vs. Measured Skid Number

$$SN = (1.38\ BPN - 31)\ e^{-.06V\ MD^{-.52}} \qquad 2\text{-}9d$$

In Equation 2-9a - 2-9d the following units must be used:

V (mph), 1 mph = 1.6 km/hr

$RMSH_{2000}$ (microinches), 1 microinch = .0254 microns

RMSH (milli-inches), 1 milli-inch = 25.4 microns

MD (milli-inch), 1 milli-inch = 25.4 microns

It should be pointed out that these relationships (Equation 2-9) do not take into account the influences of seasonal variations. However, they do establish the influence of texture on skid resistance and provide insight into the requirements of texture for adequate skid resistance. Equation 2-9d is plotted in Figure 2.6, which shows the texture required to provide skid resistance levels of 20, 30, and 50 at speeds of 20, 40, and 60 mph (32, 64 and 96 km/h).

Thus laboratory methods can be used to estimate skid numbers at any speed from representative samples of pavement systems. In designing aggregates for good skid resistance at low speeds, it is important to provide a high degree of microtexture in asperity scales below 0.5 mm (0.02 in.). For high speeds, it is also important to provide large values of root mean square texture height (or mean texture depth). For aggregates which polish to a given BPN or to a known minimum $RMSH_{2000}$ microtexture, the required macrotexture can be computed for a desired level of skid resistance at the design speed.

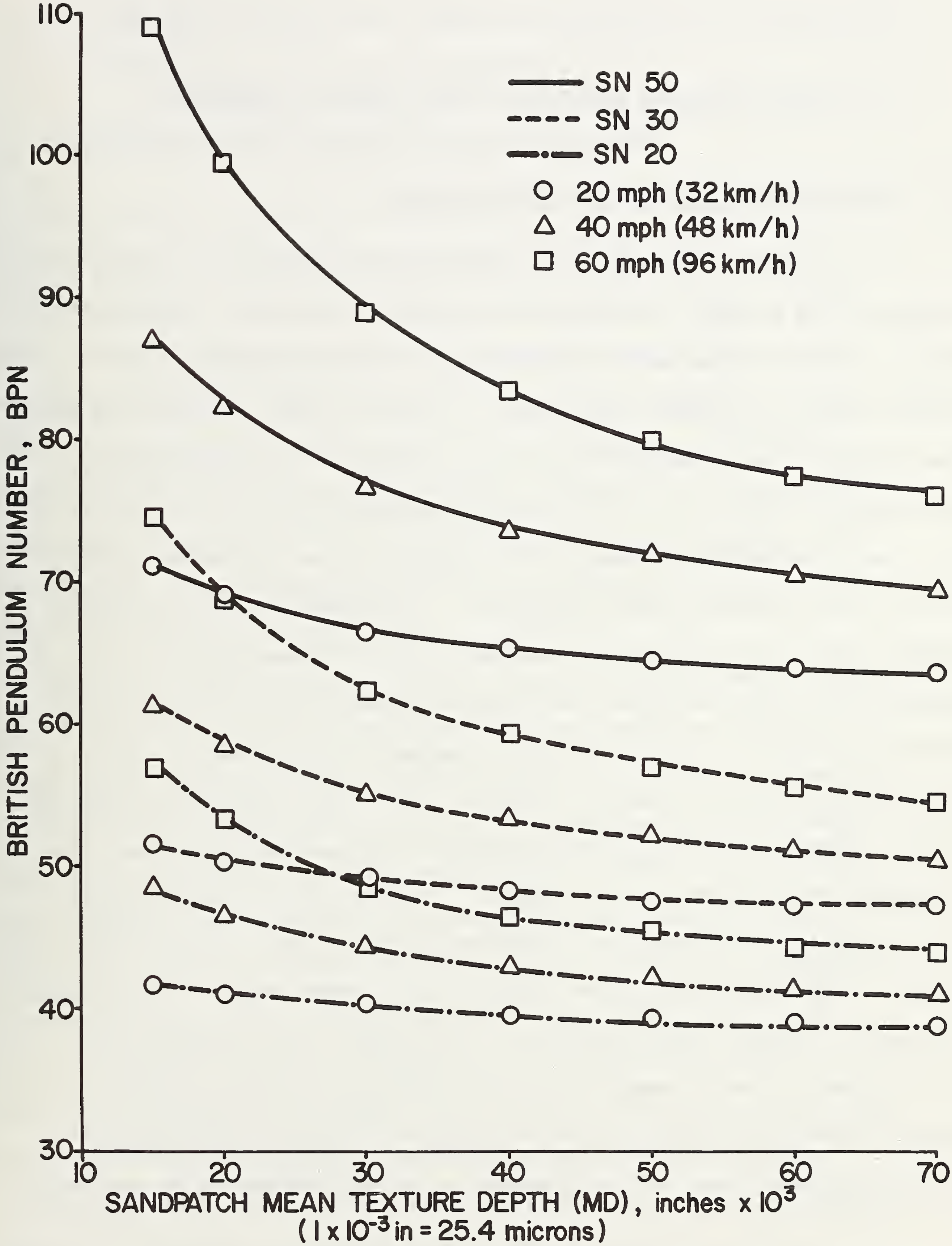


Figure 2.6. BPN as a Function of Macrotexture for Various Speeds and Skid Numbers

3.0 OPTIMAL AGGREGATE PROPERTIES TO MEET SURFACE PERFORMANCE

3.1 Role of Aggregates in Providing Performance

It is well known that aggregates provide the bulk--70 to 95 percent by weight--of the pavement surface materials [PCA, 1968; Asphalt Institute, 1974]. This fact alone makes it obvious that surface performance will largely depend on aggregate performance. In PCC surfaces, texturing and the fine aggregate, which constitutes 40 to 50 percent of the surface mortar, play the most important part in the surface performance during much of its life, until the coarse aggregate in the concrete becomes exposed to traffic, then its role acquires primary importance. In bituminous pavement surfaces, except in mixtures utilizing fine aggregate only as in sand-asphalt, it is generally the coarse aggregate that comes in contact with the vehicle tires. Therefore, characteristics of the coarse aggregate play the major role in determining surface performance [Burnett et al. 1968, Gramling and Hopkins 1974]. However, the fine aggregate portion of a bituminous mixture may also be exposed in the surface to some degree. Therefore, it is also important and its characteristics must be considered [Burnett, 1968, Gramling, 1974, Dahir 1974]. In bituminous and similar mixtures (e.g., where epoxy or other binders partially or totally replace asphalt), if the binder is well designed and properly used so as to adequately bond the aggregate particles together and hold them well in place without bleeding or flushing, the surface performance becomes almost totally dependent on the performance of the aggregate.

Therefore, aggregate properties that affect surface performance will be discussed in some detail in the following paragraphs.

3.2 Aggregate Properties that Affect Surface Performance

Most pavements to date have been made either from bituminous mixtures that physically bind aggregate particles together to provide the surface that comes in direct contact with vehicle tires, or they are made from PCC where a textured mortar surface will come in contact with vehicle tires for long periods of time, until the mortar layer is worn off exposing the concrete coarse aggregate to the surface. PCC surface aggregates include the fine aggregate portion which constitutes 40-50 percent of the surface mortar, and the top layer (1 to 2 in. or 25 to 50 mm) of coarse aggregate that may eventually become exposed and come in contact with the tire once the surface mortar layer has been worn down.

During the surface life of a pavement, particularly a bituminous pavement, surface aggregates are subjected to various stresses and influences including slow grinding through abrasion by tires, fast grinding by studded tires, crushing and breaking of edges due to high stresses caused by heavy loads (e.g., from heavy trucks) and due to physical and/or chemical actions of weathering. These influences may cause differential wear which may be beneficial in restoring surface friction, or they may result in partial disintegration of the surface. Weathering mechanisms include freezing and thawing, wetting and drying, and oxidation of some aggregate minerals, e.g. some pyrites and marcasite [Neville 1973, page 136]. Other active mechanisms include salt recrystallization in aggregate pores, hydration caused by deicing salts going into water solution, and etching or dissolution of some minerals. For example, carbonates and sulfates are etched or partially dissolved by dilute acids formed from rain water and pollutant gases (e.g., SO_2, SO_3, NO_2, and CL_2) whose concentrations have been increased in the last two decades due to increased fossil fuel emissions and industrial pollutants [Winkler 1975, PCA 1968, and Krebs and Walker 1971].

Under these influences, surface aggregates undergo varying levels of wear according to their ability to withstand these influences under the traffic and environmental conditions they may be subjected to during the life of the surface they compose. If the pavement surface is well designed and well constructed, almost any conventional paving aggregate that meets commonly used AASHTO or ASTM specifications will withstand normal surface wear. Microscopic wear could cause polishing if uniform or could improve friction, if differential. Excessive wear which could result in surface rutting and deterioration rarely occurs due to normal aggregate attrition or wearing, except in some cases where the surface may be subject to high studded-tire traffic [Preus 1970]. For this reason, many states are either prohibiting or limiting the use of studded tires on their pavements.

The most common problems caused by aggregate wear are the change in surface texture and the occurrence of wheel-path rutting. Reduction in macrotexture is detrimental to skid resistance at high speed, and rutting may allow water accumulation to depths that could increase the hydroplaning potential at highway speeds.

During the surface life, for an aggregate to withstand wear that causes detrimental changes in surface macrotexture, the aggregate must be very hard, having well-bonded grains such that it will not easily crush or fracture under traffic loading stresses, and must be free or almost free of intermittent pores that will allow water--particularly salt water--to be absorbed and retained in the pores, possibly causing harmful particle breakage due to freezing and thawing or salt recrystallization. The aggregate must also be chemically sound so as not to be subject to severe chemical decomposition due to reactions caused by reactive ingredients, pollutants, or oxidation. It may be added here, however, that differential wear due to weathering may be beneficial in restoring surface texture to polished pavement surfaces, thereby improving their skid resistance [AASHTO 1976]. If the aggregate resists excessive wear but undergoes slow differential wear, the macrotexture will be preserved, and the microtexture will be improved. Microtexture provides friction between tire and pavement through adhesion, and macrotexture facilitates water drainage in the tire-pavement contact area, and improves friction through tire deformation, known as hysteresis [Kummer and Meyer 1967]. To maintain microtextures that will provide high friction, the literature review indicates that slow, irregular crystal fracture or slow differential wear must occur at a rate that will inhibit detrimental surface polishing and loss of frictional properties without unduly wearing away the equally important surface macrotexture [Shupe 1959, page 524, Schneider 1966, page 8, and James 1968, page 9].

The aggregate role, then, particularly in bituminous pavements, is to provide throughout the surface life a macrotexture that will induce tire hysteresis (energy loss) and facilitate water drainage in the tire contact area so as to inhibit the building up of a thick waterfilm partially separating the tire from the pavement, and to provide a microtexture that will maintain a level of friction conducive to safety under prevailing conditions by inducing the tire-pavement grip that will inhibit unsafe skidding on wet surfaces during emergency braking at normal highway speeds.

There is no general agreement on quantitative scales for these textures. Different investigators have arbitrarily used different scales. In a study in the U.K., Neville [1974] found a wide range of different scales and types of texture, but concluded that most irregularities fell into three scales: Scale 1, aggregate size of 12.7 to 9.5 mm (1/2-3/8 in.); Scale 2 ranged from 250 to 100 μm; and Scale 3 texture less than 10 μm. Lees and Williams [1973] showed that the value of wet friction increases sharply when the relief of the aggregate topography rises above approximately 5 μm, and suggested limits of microtexture harshness between 10 and 100 μm within which an adequate level of wet friction may be obtained without resulting in excessive tire wear. The authors have reason to believe that about 0.5 mm (0.020 in.) may be an acceptable dividing line between macro- and microtexture, as shown in Section 2.5.1.

3.3 The Most Important Properties Affecting Aggregate Role

Important properties associated with surface aggregate resistance to wear and polishing include particle size, shape, and gradation. It is axiomatic that, to resist wear, the aggregate must be strong and tough, and to resist polishing, the aggregate must possess and retain sharp asperities. The latter condition may be attained either through differential hardness of the constituent minerals or through porosity and irregular crystal fracture. These and other important properties are related to aggregate petrography. This term includes proportion of aggregate mineral composition, mineral hardness, grain size and shape, and grain packing (vesicularity) and bonding to other grains or to the matrix. Aggregate petrography will be discussed later in more detail relative to aggregate wear and polishing.

3.3.1 Particle Size

The literature documenting research on bituminous surface aggregates generally suggests that coarse aggregate particles, which provide the macrotexture, should be in the range of minus 3/4 in. sieve (19.0 mm) to plus No. 8 sieve (2.36 mm). This range of particle size in the surface appears to provide a texture having adequate water escape channels without detrimentally reducing surface friction. Hosking [1973, page 8] found that, in the range of 3 to 25 mm, when the same aggregate was used as chippings, halving the nominal particle size increased the sideway force coefficient of friction (SFC) by about 0.08 units, provided that adequate surface texture was maintained. Mullen et al. [1974] found that when 19.0 mm and 9.5 mm nominal sizes were incorporated, respectively, in dense bituminous mixtures, surfaces having the 9.5 mm top size averaged about 8 BPNs higher than those incorporating the 19.0 mm size. Lees et al. [1976, page 41] reported that "for the closer particle spacings, the optimum size for maximum contact and the optimum size for minimal time for water to be forced out of the contact patch is in the region of 6 to 12 mm diameter." Lees et al. also suggested that inter-particle spacing should be in the range of 1 to 3 mm, depending on aggregate size within the recommended size range. Havens [1974, page 124] stated that "sand-asphalt mixtures can be designed to be as porous as the so-called open-graded plant mix seals. Particle shape and texture otherwise define skid resistance." However, Lees et al. [1976, page 41] reported that "for small particles, a maximum contact area would be achieved but also a high drainage time--a condition obviously suitable for dry and low speed conditions but unsuitable for high speed wet conditions." This statement generally reflects the thinking of the highway community, as may be surmised from the following statement in the AASHTO Guidelines [1976, page 12] regarding a high void sand mix: "A sharp sand with a high silica content is required for good skid resistance. This mix provides excellent skid resistance at lower speeds--under 40 mph or 65 kmph).

The preceding statements generally represent many other expert opinions on optimum aggregate size for skid-resistant dense and open-graded bituminous surfaces. Therefore, it appears reasonable to recommend that for optimum surface performance, coarse aggregate size should fall in the range of 3 to 12 mm, or, for practical U.S. practices, pass the 1/2-in. sieve and be retained on the No. 8 sieve.

3.3.2 Particle Shape

It has long been recognized by paving and construction researchers and engineers that aggregate shape has an important influence on pavement performance. In bituminous pavement surfaces, angular, cubical particles are preferred to rounded, flaky particles. The angular particles generally increase shear strength of bituminous mixtures through particle interlock [Marek et al. 1972] and contribute to improved wet skid resistance. Their protruding tips and asperities break the water film when the tire passes over them. As a result of a laboratory investigation on pressure distributions beneath spherical and conical shapes pressed into a rubber plane, Sabey [1959, page 545] concluded that "on individual projections on the road, it seems that pressures averaging 1000 psi (6.9 MPa) are necessary to ensure a high resistance to skidding under wet conditions, and to obtain these pressures, the individual projections should have angles at their tips of 90° or less. The results stress the importance of the shape of projections in the road surface in determining the skidding resistance of wet roads."

Havens [1974, page 124] stated that "particle size and texture define skid resistance." In a survey of the literature on bituminous paving mixtures, Benson [1970, page 20] wrote:

Moyer and Shupe [1951], in a study of the skid resistance of bituminous pavement surfaces, found that the friction values for rounded aggregate were about 25 percent lower than those for angular aggregates in wet pavement tests. Stephens and Goetz [1960] also found that the shape of the aggregate particle affects the skid resistance of a fine bituminous mix. Comparison of relative resistance values for round and angular shapes of the same material reveals an initial superiority for the angular aggregate. However, long-term skid resistance depended on the polishing characteristics of the aggregates.

To summarize, it may be appropriate to quote Benson's [1970, page 20] summary regarding the effects of aggregate shape:

> In summary, the literature indicates that the shape of the aggregate has appreciable effect on the physical properties of the mixture, on the proper asphalt content, and on the void relationship. The generally accepted principle that the shape of the coarse aggregate is critical with regard to properties of graded mixtures seems to apply only to open-graded mixtures. The literature indicates that the characteristics of the fine aggregate fraction are dominant for down-graded [sic] mixtures. Aggregate shape is also quite important in its effect on skid resistance.

3.3.3 Aggregate Gradation

It should be safe to state that all state highway departments and other agencies concerned with pavements are aware of the importance of aggregate gradation to pavement performance. Depending on the mix, a variety of gradations have been used.

It is generally well known that gradation is important for any paving mixture, but it is of particular importance in bituminous surface mixtures. The four gradings most generally used in bituminous surface mixtures are known as dense gradings, open gradings, fine gradings, and one-size gradings. Dense gradings contain materials that are well graded from coarse to fine. Because of their high stability and relatively high impermeability, they have been most widely used for surfaces that carry heavy traffic and/or are exposed to severe climatic conditions. Open gradings have been successfully used to facilitate fast drainage of wet pavements in the surface-tire contact area. In so doing, they reduce the skid resistance-speed gradient, reduce splash and spray, and reduce glare due to light reflection caused by accumulated surface water. Fine-gradings are generally used where silicious sand is readily available in large quantity, and where traffic volume and speed may be low to moderate. Under these conditions, sand-asphalt mixes appear to have given satisfactory performance relative to stability, wear, and skid resistance. One-size gradings have been mostly used as chippings in seal-coats applied to pavement surfaces to maintain resistance to abrasion and/or skidding. Different states and agencies have used different gradations under each of these four groups, depending on the performance aspect desired most and on the available materials. Typical gradations are listed in Table 3.1.

3.3.4 Aggregate Strength and Toughness

These properties are important, particularly in bituminous surfaces, because the aggregate is in direct contact with the tire. As such, the aggregate is subject to forces of shear, abrasion, and probably impact. These forces may break up the aggregate, altering its gradation, and they may abrade the particles, reducing their texture. Therefore, the surface aggregate must provide the mechanical stability and strength to resist these forces over the pavement surface life. The most commonly used test for resistance to abrasion (and impact) is the Los Angeles Test [ASTM C131]. In the United States, most aggregate specifications include the Los Angeles abrasion test requirements. Typically, not more than 40 percent loss is permitted when using this test for surface aggregates. In the United Kingdom, separate tests are used for abrasion, impact, and crushing strengths. These tests have been used to a limited extent in the United States, mostly in research, but they or similar cests may become more attractive in promoting new and

Table 3.1. Typical Aggregate Gradations

Percent Passing Sieve No. (U.S. Standard)	Size (mm)	Grading: Dense[a] (Pa. ID-2A)	Open[b] (FHWA)	Fine[a] (Pa. FJ-1)	One-Size[a] (Pa. Seal Coat)
1/2"	12.5	100	100	-	100
3/8"	9.5	80-100	95-100	100	75-100
No. 4	4.75	45-80	30-50	90-100	10-30
No. 8	2.36	30-60	5-15	60-100	0-10
No. 16	1.18	20-45	-	40-80	
No. 30	0.60	10-35	-	20-60	
No. 50	0.30	5-25	-	10-40	
No. 100	0.15	4-14	-	7-25	
No. 200	0.075	3-10	2-5	3-15	

[a]Pennsylvania Department of Transportation, Specifications, Form 408, 1976.

[b]Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP-74, U.S. Department of Transportation, Federal Highway Administration, 1974.

innovative surface aggregates. They will be discussed further in the section on "Aggregate Test Methods and Criteria."

3.4 Polish-Resistant and Wear-Resistant Aggregates

Polishing may be defined as the loss of microtexture (asperities smaller than 0.5 mm, as defined in Chapter 2), whereas wear is the loss of macrotexture (asperities and protrusions 0.5 mm and larger). Researchers have found that the mechanisms involved in wear and polishing are complex, numerous, and vary with the type of aggregate and the prevailing traffic and weather conditions. However, a review of the literature reveals that most investigators generally agree that the principal mechanisms of wear are stresses imposed by loads and environmental changes, while the principal mechanism of polishing appears to be abrasion of the small aggregate asperities due to the rubbing action when loaded tires utilize the fine road detritus as the abrasive agent. Another important mechanism is chemical change, which could induce the softening of asperities, thus hastening their loss, as in the alteration of feldspar to sericite in granite, or could induce surface roughening through differential weathering of the surface aggregates, thereby improving their frictional properties, as in arkosic sandstone. Thus, wear and polishing generally involve similar processes, which vary only in the degree and rate of material loss. Different surfaces have different requirements, depending on usage factors. Such factors include traffic loads, traffic counts (which should include some truck to car equivalency), travel speed, and environmental conditions. The type of binder and surface system must also be considered, but it may be stated that, in a general way, aggregates exposed to contact with the tire must meet similar requirements regardless of system or binder, as long as other conditions are similar.

Aggregate particle size, shape, and gradation, as well as strength and toughness, are related to aggregate wear and polishing, as was discussed in the preceding section. The most important properties that determine aggregate resistance to wear and polishing are the intrinsic properties generally referred to as aggregate petrography.

3.4.1 Aggregate Petrography

The term "petrography" is used here to refer to aggregate mineral composition, constituent mineral hardness, mineral grain or crystal size, shape, and distribution, grain or crystal consolidation, i.e., packing, bonding and cleavage, and mineral susceptibility to chemical attack and alteration.

Three important aggregate properties that have been repeatedly mentioned in the literature relative to wear and polishing resistance merit elaboration and comparison: constituent mineral proportion and hardness, grain or crystal consolidation (packing and bonding), and differential or sacrificial wear. Aggregate mineral hardness has been conventionally measured by Mohs hardness scale or more closely by Vickers or Knoop scales. The three scales are compared in Figure 3.1 and Table 3.2. In conjunction with strong consolidation of grains, aggregate mineral hardness has been found to contribute significantly to aggregate wear resistance. Stiffler [1969] found that, in a general way, the harder the mineral is, the less will be its wear rate, and that other properties such as specific gravity and melting temperature did not correlate well with the rate of wear (see Table 3.3). However, Neville [1974] reported that in substances with low melting temperatures, the flowing of the high points produces a polished surface. Hosking [1976] found that when the grains are medium to coarse (100 to 250 microns or more) and have sharp edges and angular protrusions (less than 90°) in the tire contact area, they provide good frictional resistance in addition to wear resistance. If the grains are uniform in

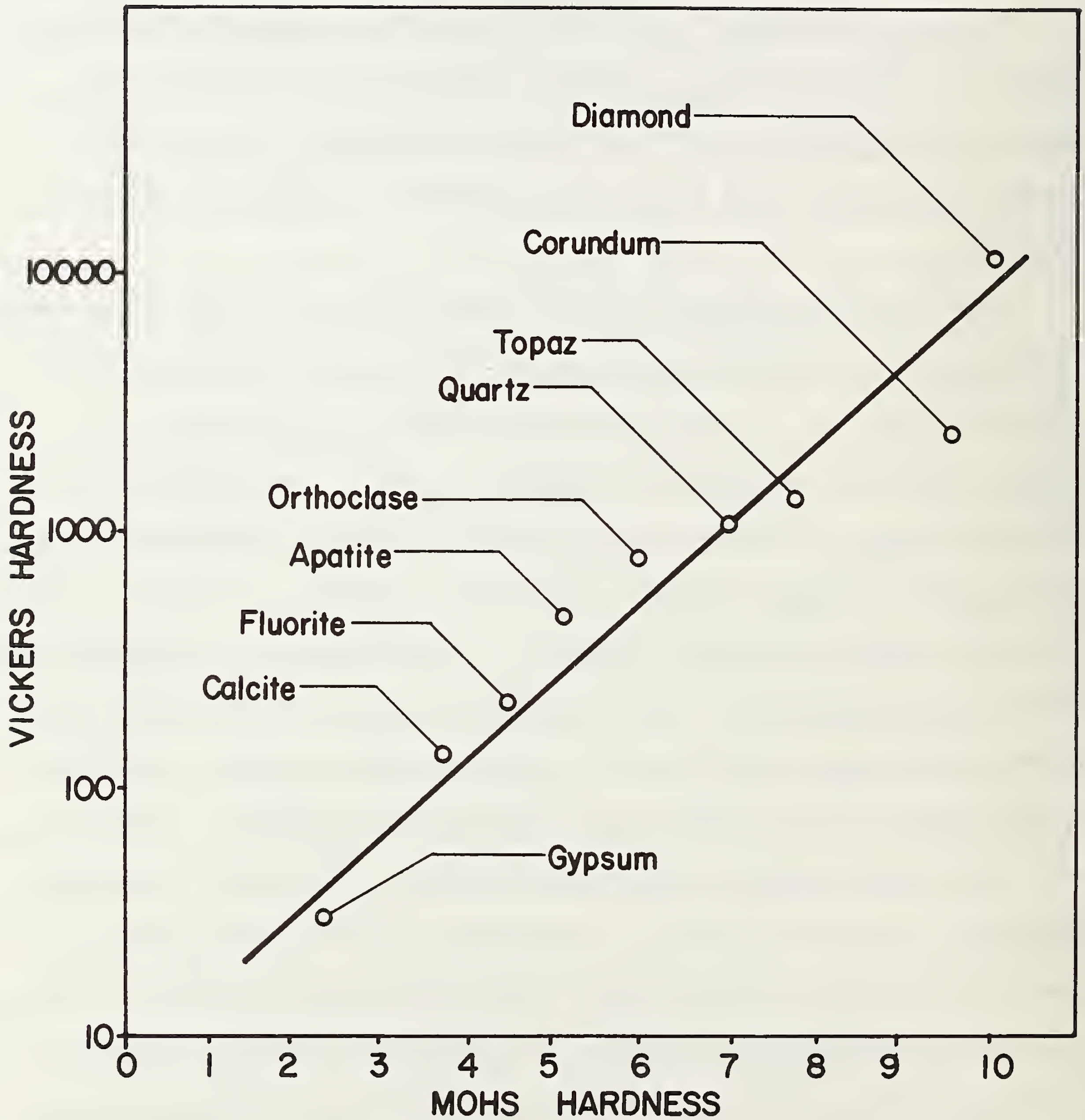


Figure 3.1. Vickers vs. Mohs Hardness [Tabor, 1954]

Table 3.2. Hardness of Various Abrasives and Minerals [James, 1968]

Mineral	Hardness	
	Mohs Scale	Knoop Number
Diamond	10	7,000
Boron carbide		2,900
Silicon carbide		2,500
Aluminum oxide (corundum)	9	2,000
Tungsten carbide		1,880
Topaz	8	1,400-1,500
Sillimanite	7.5	
Cordierite	7-7.5	
Quartz	7	800-900
Garnet group	6.5-7.5	
Olivine group	6.5-7	
Epidote group	6-7	
Chalcedony	6	
Feldspar (orthoclase)	6-6.5	600-700
Pyroxene group	5-7	
Amphibole group	4-6.5	
Apatite	5	500
Zeolites	3.5-5.5	
Fluorite	4	200
Dolomite	3.5-4	
Calcite	3	130
Gypsum	2	50
Talc	1	20

Table 3.3. Mineral Properties and Average Wear [Stiffler 1969]

Mineral	Melt Temp (°C)	Modulus[a] (psi x 10^6)	Yield Stress (psi x 10^4)	Specific Gravity	Vickers Hardness (kg/mm^2)	Abrasive Wear ($mm^3 \times 10^{-2}$) Al_2O_3	SiO_2	MgO
$CaCO_3$	825	-	-	2.70	460	42.2	30.7	13.5
CaCO (b)	825	-	-	2.70	400	38.0	40.9	12.5
Slag	1400	-	-	2.70	620	24.0	16.3	9.4
SiO_2 (c)	1700	6	-	2.10	1100	11.0	11.0	2.6
Mullite	1810	21	0.9	2.95	1720	11.0	6.5	3.1
SiO_2 (d)	1700	7	0.7	2.65	2000	7.7	6.5	1.7
MgO	2620	42	1.5	3.60	1240	3.4	1.3	0.2
ZrO_2	2650	21	2.0	5.70	1700	2.4	0.6	0.2
SiC	2200	50	5.0	3.00	4500+	0.7	0.4	0.07
Al_2O	2000	45	4.0	4.00	3300	0.3	0.7	0.03

a. psi x 10^6 = 6.89 k Pa x 10^6

b. impure

c. fused

d. crystalline

composition and/or grain size, and if they are well compacted and well cemented, they will eventually polish to smooth surfaces under the sustained polishing action of heavy traffic. Thus, hardness promotes resistance to wear but does not prevent it, nor does it guarantee lifelong surface resistance to polishing. On the other hand, if the hard crystalline grains are loosely cemented together by some binder or by a matrix softer than the crystals, the binder will lose its grip of the crystals, and the matrix will undergo relatively fast wear under the action of traffic, releasing the surface crystals before they are polished and giving way to a fresh layer of unpolished crystals. The surface renewal perpetuates the maintenance of the level of friction the aggregate surface initially provides, but may cause undesirable loss of macrotexture or even early rutting in the wheel tracks,resulting in water ponding and the problems associated with it discussed earlier (see Section 3.2).

Other attempts have been made to investigate the relationship between rates of aggregate wear and aggregate polishing. In a limited testing program, Dahir [1970] utilized a jarmill apparatus to wear and polish samples of eight natural and lightweight synthetic aggregates by tumbling them for 120 hours. He found a high linear correlation (r=0.93) between aggregate percent wear loss and frictional resistance as measured by the British Portable Tester [ASTM E 303-69]. The results are shown in Table 3.4 and Figure 3.2. This finding has recently been confirmed in another limited wear and polishing program undertaken at The Pennsylvania State University (PSU). Seven synthetic and natural aggregates used in Pennsylvania and other states were tested in 1977 for wear loss and frictional change utilizing a rubber-coated drum apparatus. The apparatus was initially used by Stiffler [1969] simply to wear aggregate particles and was later modified and used to measure the

Table 3.4. Wear, Friction, and Other Physical Properties of Eight Typical Paving Aggregates [Dahir, 1970]

Aggregate	Symbol	Bulk Specific Gravity	Water Absorption, Percent	Los Angeles Abrasion Loss (ASTM C 131), Percent	120-hr Jar Mill Wear Loss, Percent	Friction Number, BPN
Arkosic Sandstone	SS-1	2.66	2.55	N.A.	40.2	62.0
Expanded Slate	SO-1	1.58	3.50	40	31.1	60.0
Granite Gneiss	GN-1	2.67	0.41	29	22.8	54.0
Slate	SL-2	2.78	0.33	24	21.2	54.0
Limestone	LS-1	2.85	0.30	18	14.5	48.5
Granite	GT-1	2.79	0.31	36	13.0	54.0
Dolomitic Limestone	LS-2	2.87	0.40	25	10.9	47.5
Expanded Glass	SP-1	2.05	2.40	23	7.8	45.0

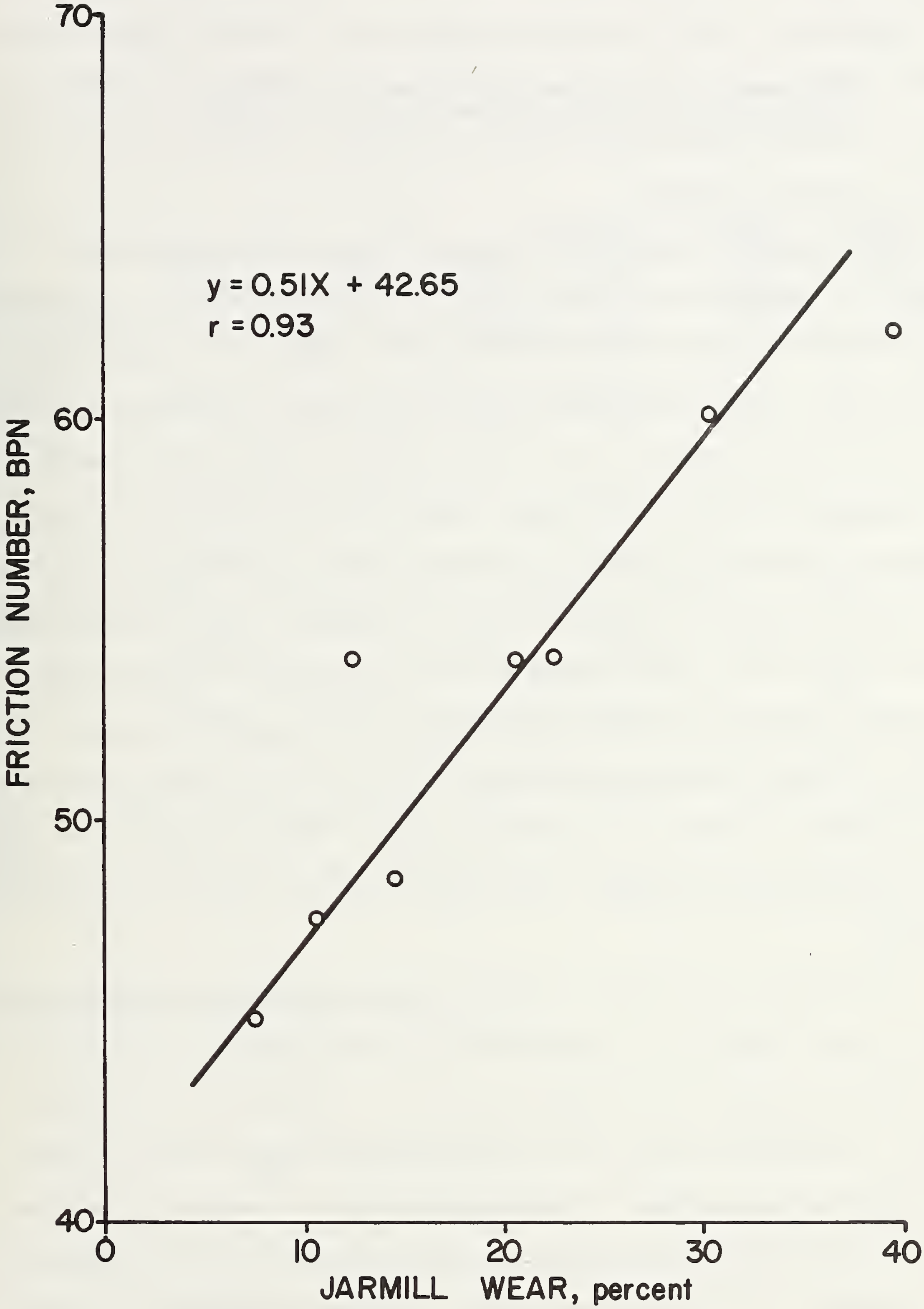


Figure 3.2. Jarmill vs. Friction Number [Dahir 1970; Dahir and Meyer 1977]

average frictional resistance occurring simultaneously with aggregate wear [Dahir, Meyer, and Hegmon 1974]. Results of the tests made recently at PSU are summarized in Table 3.5 and the correlation is shown in Figure 3.3 [Dahir 1978]. They show that for the aggregates tested, a fairly high correlation (r=0.91) exists between percent weight loss due to wear and the rate of friction loss due to polishing.

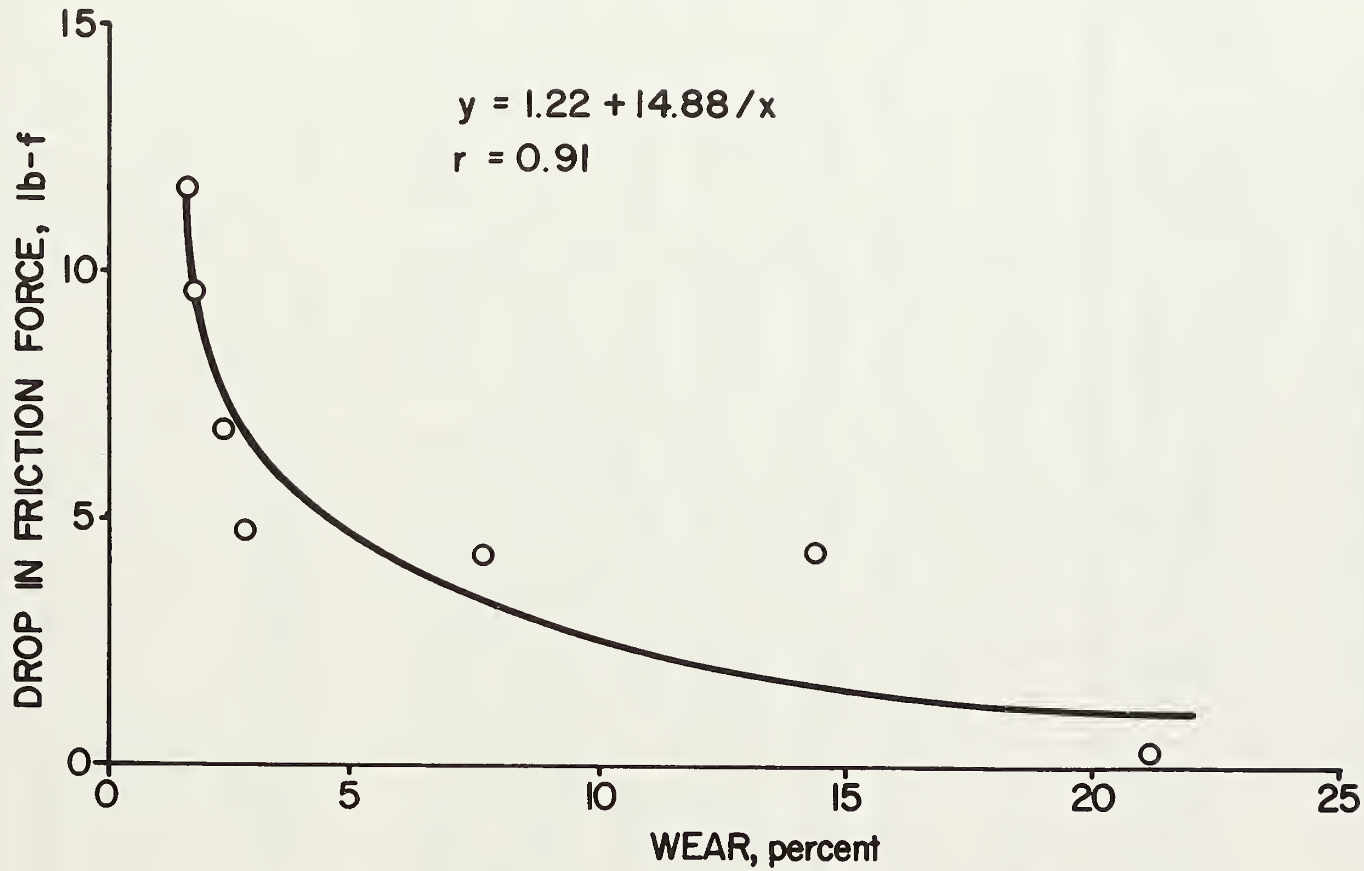
Mineral composition and differential hardness in the constituent minerals of an aggregate were found to be of the utmost importance in providing long-lasting frictional resistance [Shupe 1959, Dahir 1970, Furbush 1972, Neville 1974]. Gray and Renninger [1960] found that in carbonate aggregates, the presence of 10 percent or more hard silicious material (H=7) improves the aggregate frictional properties. Dahir and Mullen [1971] found that optimum resistance to polishing is attained when the aggregate is composed of 50 to 70 percent hard crystals (H>5) well cemented into a matrix of softer minerals (H=2 to 4). This finding was later confirmed by Mullen, Dahir and El Madani [1974] and has been substantiated by data reported recently for a study in Maryland [Smith and Morawski 1977]. Data from these studies have been combined in Figure 3.4. In these studies, BPN measurements were made on laboratory-polished specimens. Data points from each study are readily identifiable.

The hardness of minerals of which paving aggregates are composed varies in the range of H=2 to 9. Most naturally-occurring aggregate material is composed of minerals in the range of H=2 to 7. Arbitrarily, but rather widely, H=5 has been used as the dividing number between minerals termed soft and those termed hard. A differential of 2 to 4 degrees of hardness has been found effective in imparting long-lasting frictional resistance to the aggregate. Other things being equal, a higher differential in hardness produces a higher and a more lasting friction [Tourenq and Fourmaintraux 1971]. However, the level at which the differential hardness occurs is also important,

Table 3.5. Results of Wear and Polishing of Seven Aggregates Using Penn State Small Drum Machine [Dahir 1978]

Aggregate	Initial Weight g	Final Weight g	Percent Weight Loss	Initial Friction Force lbf[a]	Final Friction Force lbf[a]	Drop in Friction Force lbf[a]
Juniata, Pa. Red Bed	20.8	17.8	14.4	16.99	12.56	4.43
Maryland Granite	19.8	18.3	7.6	9.97	5.73	4.24
Texas Bed Rock	17.0	16.5	2.9	15.88	11.08	4.80
Connecticut Trap Rock	18.8	18.5	1.6	19.58	7.76	11.82
Expanded Shale	22.6	17.8	21.2	20.32	20.30	0.02
Blast Furnace Slag	16.4	16.1	1.8	18.47	8.87	9.60
Fused Refuse	18.6	18.2	2.2	14.59	7.76	6.83

[a]A pound-force (lbf) = 4.45 Newtons (N)



(1 lbf = 4.45N)

Figure 3.3. Rotating Drum Wear Versus Drop in Friction Force [Dahir 1978]

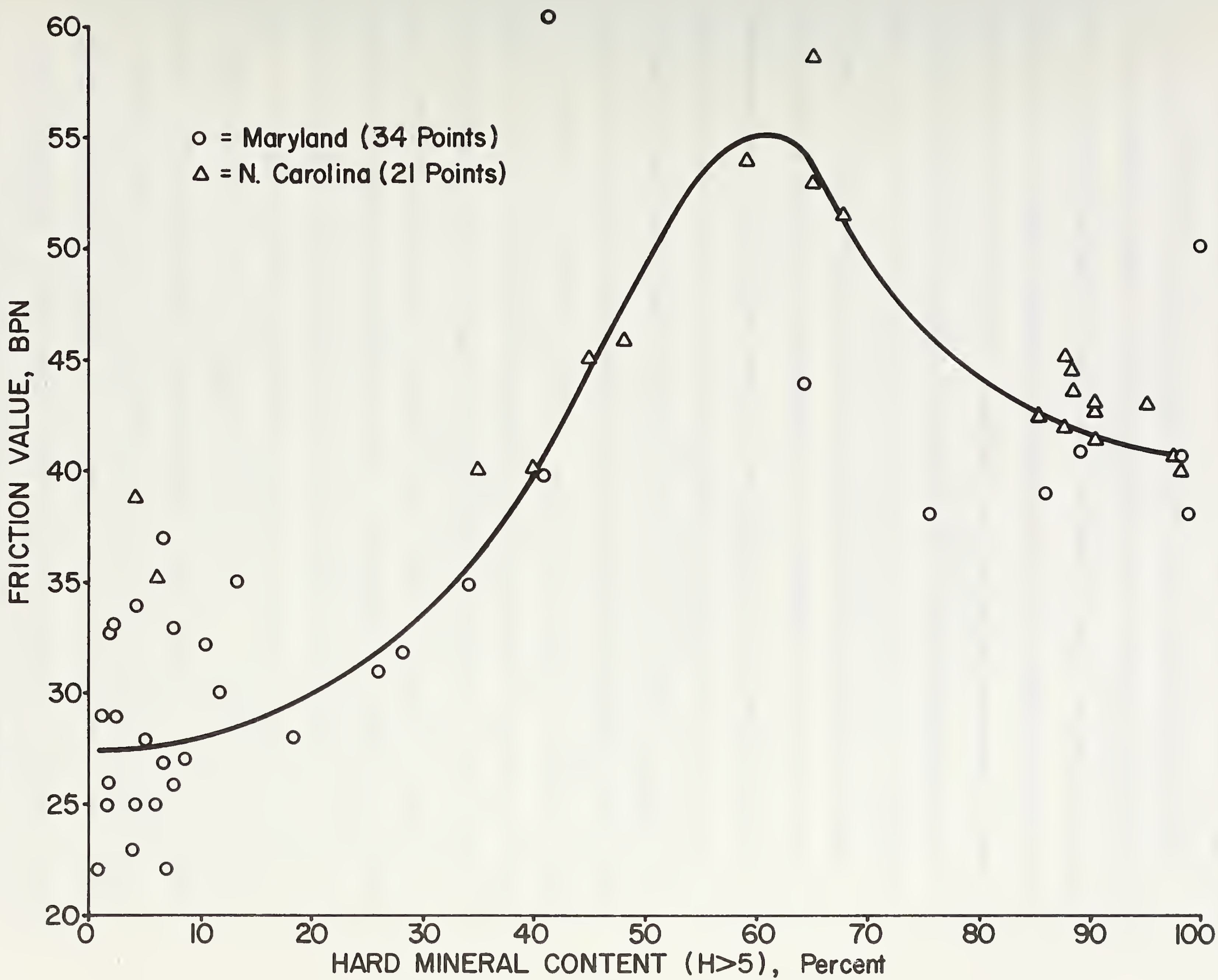


Figure 3.4. BPN Friction Values (ASTM E 303)Versus Hard Mineral Content [Dahir 1971, Mullen 1974, Smith 1977]

because of the non-linearity of Mohs' scale, as may be seen from Figure 3.1 and Table 3.2.

Neville [1974] reported that the artificial aggregate calcined bauxite, which is reputed to possess high resistance to both polishing and wear, is composed of corundum crystals (H=9) held in a brown glassy matrix (H≃6), and that mullite (H=6 to 7) is often present in calcined bauxite. The proportions of the constituents were not reported; however, Hosking [1970] showed that 20 to 40 percent hard grit in artificial aggregate produces high friction numbers.

Based on this information and the reputation of calcined bauxite, one may conclude that if an aggregate can be produced to contain 20 percent or more crystals of very high toughness and hardness (H=8 to 9), strongly cemented to a matrix of softer albeit hard material (H=6 to 7), it is very likely that a high degree of wear resistance will be provided through hardness and good bonding, while polish-resistance will be provided through the slow differential wear of the constituent materials. It must be remembered, however, that crystal size, shape, density of distribution and manner of fracture are also important companion properties that influence polish resistance.

Another satisfactory approach to providing high-performing surface aggregate has been to use aggregates made from very hard crystals well consolidated but interrupted by many small, well-distributed voids (porosity of 25 to 35 percent of volume). If the bonding of the hard crystals is strong and the manner of crystal fracture results in slow, sacrificial wear that continually provides sharp, irregular crystal edges, the surface will remain harsh, providing adequate polish resistance while the slow wear contributes to a long surface life. Examples of this type of aggregate include calcined bauxite and some air-cooled blast-furnace slags. These and other artificial aggregates are discussed in more detail in Chapter 5.

Several researchers in this country and some foreign countries (particularly the British Transportation and Road Research Laboratory) have done considerable research on surface aggregates in the past two decades. Based on his experience with variations in the resistance of different aggregates to polishing, one researcher [James 1968] suggested that an ideal aggregate that is likely to perform well in polish resistance could fall into one of five theoretical models of polish-resistant aggregates. James' models are shown in Figure 3.5. Examination of the models leads to the conclusion that, at the present level of technology, the most feasible aggregates (technically and economically) may be obtained either by dispersion of extremely hard (Mohs H=8 to 9) particles in a relatively softer matrix (H=6 to 7), such as dispersing fused alumina or silicon carbide in mullite, or in a hard glassy matrix, or by calcining very hard materials (H=8 to 9) so that they will fracture at a slow rate in an irregular manner, as is the case in calcined bauxite and some calcined flint. Extremely hard (Mohs H=9 to 10) crystalline materials of uniform mineral composition, which could resist wear and could be cut in angular shapes that would retain pavement friction through expected surface life, are not available at the present time in large quantities at economic costs. Examples of these materials include diamond, aluminum oxide and silicon carbide. In addition to low availability and high cost, there is no guarantee that such materials would not polish under traffic impoundment, that they would be sufficiently crush-resistant in conglomerations of crystals, and they would not create high levels of noise and tear up tires to such an extent as to be generally unacceptable to the driving public.

Porous aggregates (James, fifth model) may be a viable solution if common materials such as clay could be economically made into vesicular

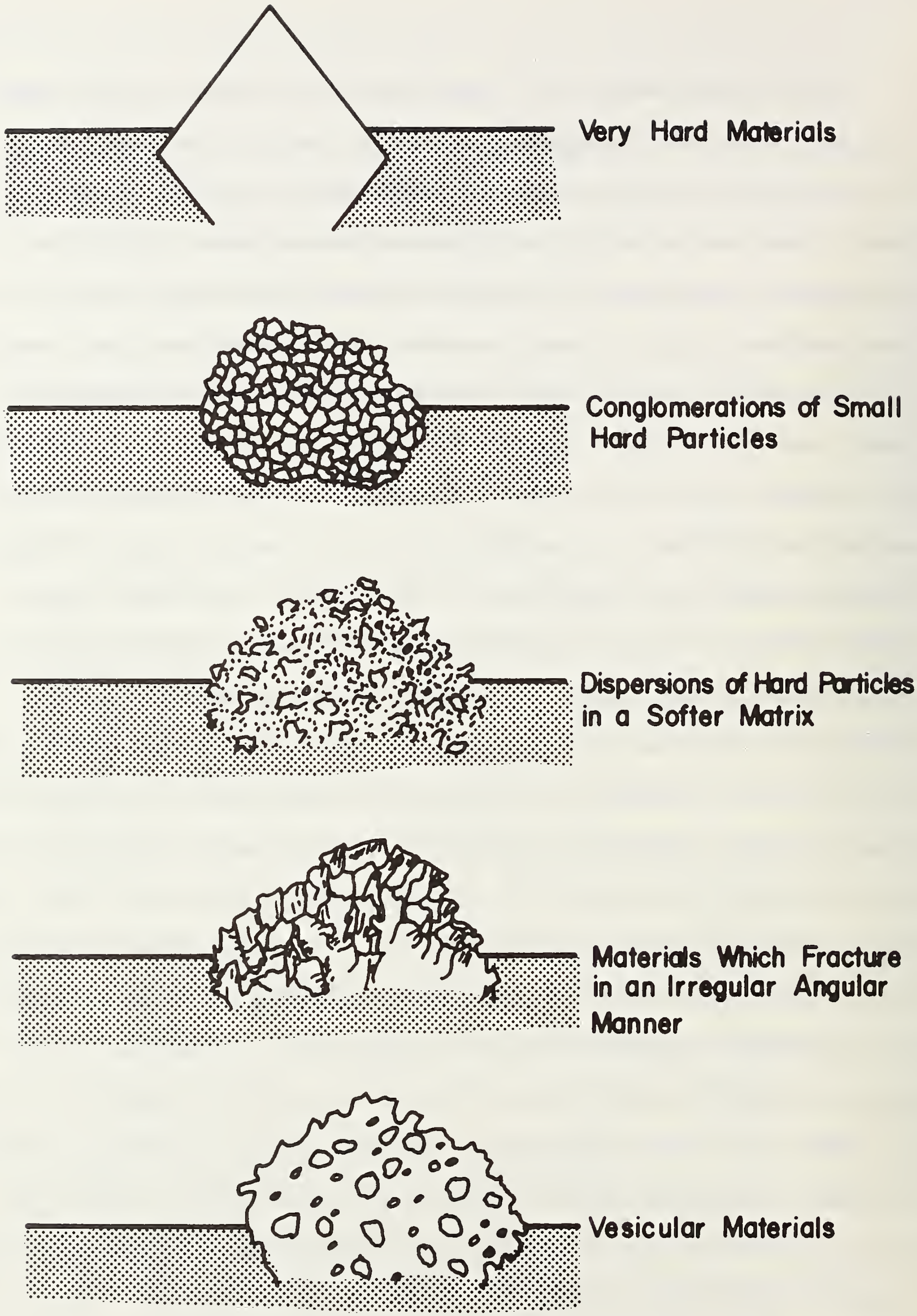


Figure 3.5. Some Theoretical Models of Polish-Resistant Aggregates [James 1968]

synthetic aggregates with high hardness (H=8-9) and high strength ($AAV \leq 10$ - see Section 3.8.1.1). According to Hosking [1976], a laboratory experiment at the Transport and Road Research Laboratory showed promise for at least one such material used in "Mössite" refractory bricks. In the experiment, it was found that for the resulting material to meet high requirements of friction and strength, optimum size pores in the aggregate should be about $200 \pm 50\mu$ and porosity about 30 ± 5 percent. However, such material has not been produced in quantity nor has it been tested in the field. Processes to modify and synthesize aggregates for surface mixtures are discussed at length in Chapter 5.

In summary, the level of aggregate performance required for a given situation should be considered in the light of surface performance requirements for that situation. Traffic, roadway geometry, environment, accident occurrence, cost, and other performance parameters (noise, tire wear, etc.) should all be considered. Thus, it would be prudent to use a readily available aggregate if it has been field tested and found satisfactory under prevailing conditions. History of aggregate performance may be the best guide for its use. In the case of a new aggregate with no performance history, or a requirement that has not been met by available aggregates, selective laboratory tests (Section 3.6) should prove helpful as guidelines. A system for the selection of aggregates to meet surface requirements can be developed by any highway department based on a combination of tests and experience. One such system has been developed in Pennsylvania. It is known as the Skid Resistance Level (SRL) system [Gramling 1976]. In this system, aggregates are placed in five categories and rated according to their skid resistance performance when used as the coarse aggregate in bituminous surfaces. Skid resistance level requirements depend on the average daily traffic using the pavement. The Pennsylvania ratings are summarized [Gramling 1976] as follows:

E = Excellent - Sandstones, gravels (mostly sandstone or siltstone) with less than 10 percent carbonate particles.

H = High - Siltstones, argillites, some quartzites, basalts, gneisses, granites, blast furnace slags, gravels with 10 to 25 percent carbonate particles.

G = Good - Most quartzites.

M = Moderate - Carbonates of Cambrian age, serpentine.

L = Low - Carbonates younger than Cambrian.

Each aggregate source in the state is given a letter rating based on the performance history of the aggregate and on field and/or laboratory tests to verify the aggregate petrography, polish resistance, and wear resistance. The rating is reviewed periodically and may be revised if subsequent data warrants the revision.

Gramling [1976] explains application of these ratings as follows:

> Each wearing surface advertised for bids has a coarse aggregate requirement for SRL dependent upon current average daily traffic for resurfacing, or anticipated initial daily traffic on new facilities, as shown in the following table.

Table 3.6. Aggregate SRL Requirements

ADT	SRL
20,000 and above	E
5,000 to 20,000	E, H, Blend[a] of E & M or E & G
3,000 to 5,000	E, H, G, Blend of H & M or E & L
1,000 to 3,000	E, H, G, M, Blend of H & L or G & L or E & L
1,000 and below	Any

a. All blends are 50 percent by weight.

Obviously, the availability of aggregates to meet a given performance level will vary from location to location. Whatever the available aggregate may be, it could be placed in a system such as the one just described. Generally, a problem arises when aggregates to meet the highest levels of performance are lacking or in short supply. In these circumstances, it may be desirable to "manufacture" aggregates to meet the requirements by using them as the coarse aggregate or by blending them with available, less expensive aggregates. Manufactured aggregates, often referred to as artificial or synthetic aggregates, have been in use for many years, and they include a variety of materials such as expanded shale, slate or clay, expanded blast furnace slag, calcined bauxite, crushed bricks and ceramic materials. However, efforts to improve the performance and reduce the production costs of synthetic aggregates are continuing. This is the subject of Chapter 5.

One nagging problem has been to produce high quality aggregates which will resist both wear and polishing. Based on experience reported in the literature, and partially on speculation based on experience and knowledge of aggregate and pavement surface properties and performance, a table has been prepared which includes desirable target aggregate property values. Table 3.7 [Dahir 1978] includes suggestions for aggregate property values that would be expected to result in high resistance to both wear and polishing. It should be recognized, however, that although these values are exhibited by available aggregates, no single aggregate achieves all these values. These values are suggested as guidelines for developing new aggregates, although it may be unlikely that any single aggregate will attain all these values.

Table 3.7. Target Values for Properties That Would Enhance Aggregate Skid Resistance and Wear Resistance

Property	Value Range	Reference
Mohs Hardness of Hard Fraction	8-9	James 1968, Neville 1974, Hosking 1976
Mohs Hardness of Soft Fraction	6-7	James 1968, Neville 1974
Differential Hardness, min.	2-3	Shupe 1959, Burnett 1968, Dahir 1971, Furbush 1972, Neville 1974
Percent of Hard Fraction		
Natural Aggregate	50-70	Dahir 1971
Artificial Aggregate	20-40	Hosking 1970
Hard Grain or Crystal Size	150-300 μm; Avg. 200	Hosking 1970 and 1976
Hard Grain or Crystal Shape	Angular Tips (≤90°)	Sabey 1959, Hosking 1970, Dahir 1971
Percent Porosity (Vesicularity)	25-35	Hosking 1974 and 1976
Pore Size, Optimum	125 μm	Hosking 1976
Aggregate Particle Size Range	3-13 mm	Szatkowski 1972
Aggregate Particle Shape	Conical, Angular (≤90°)	Sabey 1959, Hosking 1976
Los Angeles Abrasion, Percent	≤20	Dahir 1971, Howe 1976
Aggregate Abrasion Value, Percent[a]	≤8	James 1968, Hawkes 1972, Neville 1974, Hosking 1976
Aggregate Impact Value, Percent[a]	≤20	James 1968, Hosking 1970, Hawkes 1972
Polished Stone Value, BPN[b]	≥75	James 1968, Hawkes 1972, Neville 1974, Hosking 1976

[a]According to British Standards Institution, BS 812:75.

[b]According to BS 812:75 or ASTM D 3319-74T and E 303 (using auxiliary scale).

3.5 Aggregate Test Methods and Criteria

Until recent years, most U.S. available test methods and criteria for aggregates generally consisted of methods that were not initially intended for surface aggregates per se, but for paving aggregates in general. However, since pavement surface performance has become of major concern to those who must design, construct, and maintain the surfaces, available test methods applicable or related to surface aggregates have been used to evaluate these aggregates (e.g., Los Angeles Abrasion Test, ASTM C 131). Other test methods have been improvised and used by various individual agencies such as various polishing procedures [Goodwin 1969, Mullen 1971, Dahir 1976], and the insoluble residue tests for carbonate aggregates [ASTM D 3042, Furbush and Styers 1972]. Other methods have been borrowed from foreign agencies concerned with the problem (e.g., the British Wheel Test, ASTM D 3319-74T). In view of the relatively recent innovations in improving or improvising aggregates for improved surface performance, there is no complete set of tests which has acquired universal acceptance, and efforts to improve current test methods or initiate new methods continue. Some of the currently used methods may have proven useful to some agencies which have related surface performance to the tests they have used; therefore, they find it expedient to continue using these methods though other agencies may not find them as useful for their purposes. Selected methods and criteria that to date have been used in the U.S. extensively or appear to have some wide acceptance are summarized in the following paragraphs under two headings: (a) General Aggregate Quality Tests and (b) Aggregate Polish-Susceptibility Tests.

3.6 General Aggregate Quality Tests

3.6.1 The Los Angeles Abrasion Test (ASTM C 131)

This is the method most commonly used in the United States to test the ability of aggregates to withstand abrasion. Although the scope of the test method as written does not include impact effects, nevertheless the tested aggregate is subjected to impact forces. Therefore, resistance of the aggregate to impact is also implied. In this test, a sample of specified quantity (normally 5000 g) and specified gradation of coarse aggregate (retained on #8 sieve) is placed in a hollow steel cylinder, 20 in. (500 mm) long and 28 in. (700 mm) in diameter and having a 20-in. (500 mm) long inner steel shelf protruding radially 3.5 in. (90 mm) into the cylinder. Twelve steel spheres each weighing 417 g are placed with the aggregate sample and the cylinder is rotated mechanically for 500 revolutions at a speed of 30 to 33 rpm. The aggregate is then washed over a No. 12 sieve (1.68 mm opening), dried, and weighed. The percentage loss by weight of the original sample is used to determine whether the aggregate tested meets required specifications. Typically, most states that use this method specify 40 percent loss as the maximum acceptable level for surfacing aggregates. It may be added that this method has been employed for aggregates used in both PCC and bituminous mixtures and that modified procedures and sample sizes have also been used. Some of these variations are discussed in ASTM Method C 131 and elsewhere in the literature.

3.6.2 Sodium Sulfate or Magnesium Sulfate Soundness Test (ASTM C 88)

This test is used to determine the potential resistance of the aggregate to weathering. In this test, a sample of about 5000 g of aggregate having a known sieve analysis is immersed in a solution of either sodium or magnesium sulfate for 16 to 18 hours. The sample is then dried to a constant weight at 230° F (110° C) and immersed again in the solution. The process is repeated for 5 or 10 cycles, then the sample is cooled, washed, dried, and sieved, and the percentage of weight loss is determined. Particle disintegration resulting in a weight loss in the sieved sample presumably occurs due to enlargement of salt crystals in aggregate pores, thus simulating the action of freezing water. According to some researchers [Krebs and Walker 1971], the test method often appears to be unreliable for distinguishing sound from unsound aggregate. However, in the absence of performance history it may give a useful indication of the expected aggregate resistance to freeze-thaw and salt recrystallization effects. Alternate methods of testing in common use are the AASHTO Freeze-Thaw Test for Aggregates, T103-62, and the Sodium or Magnesium Sulfate Soundness Test, T104-65.

Some researchers feel that the best way to test coarse aggregate to be used in PCC is to prepare concrete samples using the aggregate and test these samples using ASTM Designation C682-75.

3.6.3 Petrographic Analysis (ASTM C 295)

This test method is recommended for testing aggregates to be used in PCC. The objectives of this test are (a) to describe and classify the mineral constituents of the sample, and (b) to determine the physical and chemical properties of the constituent minerals and their influence on the quality of the material for its intended use.

Petrographic examination can provide useful fundamental information and can help determine what other tests should be run on the aggregate.

In recent years, it has become one of the primary tests recommended for surface aggregates to be used in bituminous surfacing, and in recognition of this fact, ASTM has recently organized a task force in subcommittee D04.35 to look into the feasibility of devising a petrographic analysis test method for bituminous surface aggregates.

3.6.4 Scratch Hardness Test (ASTM C 235)

This is a test for classifying coarse aggregate particles 9.5 mm or larger (retained on the 3/8 in. sieve) as having a hardness greater or less than a brass rod 1/16 in. (1.6 mm) in diameter having a hardness between that of a copper penny (U.S. Lincoln design) and that of a nickel (U.S. Jefferson design). If the particle is scratched, it is classified as "soft." A scratched aggregate particle made from more than one type of mineral is considered soft only if the soft component is one third or more of the volume of the particle. The percentage of soft particles in the sample is reported to indicate the average softness (or hardness) of the sample.

The usefulness of this test in classifying coarse aggregates as to their suitability for surfacing is controversial. The petrographic analysis is thought to be a superior approach in classifying the coarse aggregate.

3.6.5 Organic Impurities in Concrete Sands (ASTM C 40)

This is a test to determine whether organic compounds injurious to mortar or concrete are present in natural sands to be used in the concrete. The test consists of shaking a sample of sand vigorously in a 3 percent solution of sodium hydroxide in a graduate and allowing it to stand for 24 hours. Then the color of the solution is compared to a solution of sodium dichromate in sulfuric acid; if it is darker, the sand has organic impurities.

If impurities are indicated, other tests, such as the mortar strength test, should be run to determine if impurities are deleterious.

3.6.6 Coating and Stripping of Bitumen Aggregate Mixtures (ASTM D 1664)

This test is intended to determine the retention of a bituminous film by an aggregate in the presence of water. The aggregate is coated with the bitumen at a specified temperature appropriate to the grade of bitumen used. The coated aggregate is cured and then immersed in distilled water for 16 to 18 hours. At the end of the soaking period, the total area of the aggregate on which the bituminous film is retained is estimated visually as above or below 95 percent.

3.7 Aggregate Polish Susceptibility Tests

During the past two decades, several polish susceptibility tests have been proposed and used by various investigators. However, progress towards acceptance and universal use of these methods has been slow because many questions remain unanswered. Some methods that have either been standardized or are being considered for standardization by ASTM are briefly discussed in the following paragraphs.

3.7.1 Insoluble Residue in Carbonate Aggregates (ASTM D 3042)

This test is used to determine the percentage and gradation of insoluble material in carbonate aggregates intended for use in pavement surfacing. The implication is that the more insoluble material contained in the carbonate aggregate, the less will be the susceptibility of the aggregate to polish. However, it has been shown (Figure 3.4) that only hard mineral grains (H>5) will resist polishing. The Pennsylvania method of testing for insoluble residue, PTM 618 [Furbush and Styers 1972] recognizes this fact and specifies that the silicious and hard mineral grains should be separated from the non-silicious and soft grains, and that the percentage of each be determined by the point counter method. This feature makes PTM 618 a better procedure.

Based on limited data of insoluble residue determinations, Furbush and Styers [1972] found that a minimum of 20 percent hard insoluble residue of plus No. 200 sieve size ($\geq$0.075 mm) is required for a skid number of 40 at 40 mph (64 kmph). For high skid resistance values on heavily traveled pavements, 50 to 70 percent hard mineral content in natural aggregates is recommended (Figure 3.4 and Table 3.7).

The ASTM D3042 test consists of placing 500 g or more of the aggregate passing the 3/8 in. (9.5 mm) sieve and retained on the No. 4 (4.75 mm) sieve in 1000 ml of 6N HCL acid solution in a container and agitating the mixture until all the carbonate material has reacted with the acid, leaving in residue the non-carbonate particles. The gradation and percentage of residue is calculated and reported as insoluble residue content. The size as well as the hardness of the insoluble particles will influence aggregate skid resistance performance. Coarse, hard particles improve skid resistance. Fine particles, passing the No. 200 sieve, may be of dubious value. If they consist of hard silica, they could help to improve skid resistance; but if they consist of soft, clayey material, they may have an adverse effect. Since it is difficult to identify the material passing the No. 200 sieve, it is often not considered in the insoluble portion.

3.7.2 Accelerated Polishing of Aggregates Using the British Wheel (ASTM D 3319)

This test was borrowed from British Standards BS 812, and was modified and standardized as a tentative procedure to estimate the extent to which different coarse aggregates may polish. In this test, coarse aggregate particles passing the 1/2-in. (12.5 mm) sieve and retained on the 3/8-in. (9.5 mm) sieve are placed in strong bonding material (e.g., epoxy) in a curved metal mold forming a specimen 3.5 by 1.75 by 0.63 in. (88.90 by 44.45 by 16.0 mm), and tested for initial friction using the British Portable Tester (ASTM E 303). Fourteen such specimens in sets of four for each aggregate to be tested and two control specimens of a standard aggregate are clamped around the periphery of a cylindrical wheel made to hold the specimens tightly in place. The wheel is then rotated at about 320 rpm against a pneumatic-tired wheel that bears on the aggregate specimens at a loading of about 88 lb_f (391 N) inducing aggregate surface polishing. The polishing action is aided by adding separately equal weights of No. 150 silicon carbide and water, each fed at the rate of 25 g/min. The rotation is continued for 10 hours, followed by cleaning the samples thoroughly with water and testing each for the average friction of the sample particles. Friction testing is made with the British Portable Tester (ASTM E 303) according to the modified procedure intended for this test. Friction values of the specimens of the same aggregate samples are averaged and reported as the Polish Value (PV) of the aggregate. This procedure was modified from the British Polished-Stone Value (PSV) laboratory test (BS 812) using the same apparatus and procedure except that grit size and rotation time have been adjusted for what is thought to be a more efficient operation. Values of aggregate PSV have been correlated with field sideway force coefficients (SFC) in Britain [Szatkowski and Hosking 1972]. In the U.S., some states (e.g., Texas and Pennsylvania) are using the British Wheel and Portable Tester for rating surface aggregate performance [Hankins 1974 , Gramling 1976]. It should be noted that the ASTM and Texas versions of procedure do not use the auxiliary scale on the BPT which is specified in the British Standard; thus the values reported are equal to six tenths (0.6) of the British PSV.

3.7.3 Other Accelerated Polishing/Wear Laboratory Methods

Several other methods have been used in the United States to evaluate the polishing and wear characteristics of aggregate or pavement surfaces in the laboratory [Goodwin 1969, Mullen 1971, Dahir 1974, ASTM Book 15, 1977].

Some of these methods have been standardized by ASTM while others are being considered for standardization. Some of the methods are:

a. Circular tracks utilizing full-scale tires to wear and/or polish relatively large samples (larger than 6 in., 152 mm, in diameter). These tracks provide the nearest simulation of field wear and polishing conditions, but they are fairly expensive to build and operate. Such tracks have been used in Arkansas, Maryland, and Texas.

b. Proposed Recommended Practice for Laboratory Accelerated Polish Test for Aggregates and Pavement Surfaces Using a Small Diameter Pneumatic Wheel Circular Track. This method is under consideration in ASTM Committee E-17 on Skid Resistance. It has been used in Arizona, Arkansas, Kansas, North Carolina, and Pennsylvania.

c. Tentative Recommended Practice for Accelerated Polish Test for Frictional and Polishing Characteristics Using a Small Torque Device, ASTM E 510. This method is now being used in Georgia and Mississippi.

d. Standard Recommended Practice for Testing Pavement Polishing in the Laboratory (Full-Scale Wheel Method), ASTM E 451. The usefulness of this method is limited by the fact that polishing wear is imparted solely by a sliding action that does not simulate normal tire rolling on highway pavements. This method has been used in Florida and Tennessee.

e. A "portable variable-speed skid resistance tester," developed at North Carolina State University, is being considered as a recommended practice by ASTM Committee E-17.

f. A reciprocating rubber-pad pavement polishing machine, which has been used at The Pennsylvania State University, has been modified recently to a portable apparatus capable of polishing aggregate or pavement samples in the laboratory or on the highway.

These and other polishing methods, which have been used by various agencies, have proved useful for ranking materials with respect to susceptibility to wear and polish, but they have not been able to predict accurately the effects of actual traffic and weathering unless they are supplemented with field performance correlations. In most of the methods, polishing is measured by the resulting friction number obtained with the British Portable Tester (ASTM E 303), which is mainly sensitive to surface microtexture.

3.8 Evaluation of Laboratory Aggregate Test Methods

Each of the test methods discussed gives some satisfactory indication of the quality of aggregate in the area for which the method was designed. However, some of these methods have proved more useful than others. Also, in most if not all cases, the methods were designed to test naturally occurring mineral aggregates. Now that the need and viability of using synthetic aggregates have become established, some of the available test methods need to be improved or modified, and in some cases new methods need to be designed. For example, the Los Angeles Abrasion Test Method (ASTM C 131) needs to be modified or a substitute method provided for testing abrasion, crushing and impact separately rather than having their combined effects lumped together. Furthermore, this method is not well suited for testing lightweight (highly porous) aggregates. Accordingly, modifications have been proposed for testing lightweight synthetic aggregates [Ledbetter 1971]. The Scratch Hardness Test (ASTM C 235) does not provide sufficient information on the important property of surfac- aggregate hardness and needs to be replaced by a more comprehensive hardness test--perhaps as part of a petrographic analysis method to be applied to surface aggregates. Petrographic Analysis, currently recommended by ASTM (C 295), was designed for Portland cement concrete aggregates only and does not meet all the needs of analyzing surface aggregates, particularly for bituminous mixtures where the part played by aggregates in surface performance is even more important than in the case of PCC.

A polish susceptibility test that would be applicable to both aggregate and pavement mixtures would also be desirable because correlations of test results on both the aggregate and the mixture incorporating it would be more meaningful. Furthermore, if a test less cumbersome than the British Wheel and one that utilizes larger specimens is found, it would receive more acceptance, and would be more expedient and meaningful.

In reviewing the great mass of literature available on aggregate performance in pavement surfaces, it appears that if the aggregate is to give high performance, it must meet three basic criteria: (a) resistance to wear, (b) resistance to polishing, and (c) resistance to weathering. Resistance to wear implies resistance to abrasion, crushing and impact forces. Ostensibly, the Los Angeles Abrasion Test was designed for naturally occurring aggregates with all these factors in mind, but in view of the proven viability of some synthetic aggregates and the continuing search for others that will meet surface requirements, a separate method should be used to test each property that contributes to wear resistance. British Standards, BS 812 [1975] includes such procedures, which can be used until better methods are found. To test resistance to polishing, several laboratory techniques have been designed and used for the purpose, as has been discussed earlier. No single method has received universal acceptance, perhaps because most if not all of these techniques are rather cumbersome and may give erratic results. It appears that the time has come to develop (or attempt to develop) a universal procedure that could be more expedient and more efficient. Thus far, attempts in this direction include combining aggregate petrographic analysis with some field or laboratory skid resistance test, as has been practiced recently by PennDOT [Howe, 1976]. Resistance to weathering is perhaps the most difficult property to determine in the laboratory because not all weathering factors are known, and for those that have been determined, the degree to which each factor contributes to the weathering process has not been quantitatively determined. However, the methods already discussed, such as the sulfate tests (ASTM C 88), Petrographic Analysis (ASTM C 295) and the Insoluble Residue Test for Carbonate Aggregates (ASTM D 3042), are useful though not completely satisfactory because they are still cumbersome and sometimes erratic. Until better methods are found, a combination of the currently used methods should be continued as long as it covers the three major properties--resistance to wear, polishing, and weathering. Three British test methods for abrasion, impact and crushing are briefly summarized, and an example of a procedure utilizing current methods is suggested.

3.8.1 Summary of Three British Test Methods

3.8.1.1 Determination of Aggregate Abrasion Value (AAV)

This test gives a measure of the resistance of aggregates to surface wear by abrasion. In this test, clean aggregate particles passing the 1/2-in. (12.7 mm) and retained on the 3/8-in. (9.52 mm) sieves are glued to a flat metal tray 3-5/8 x 2-1/8 x 5/15 in. (92 x 54 x 8 mm). The preweighed tray holding the particles is rotated 500 revolutions at a speed of 28-30 rpm against a flat circular iron or steel lap not less than 2 ft. (60 cm) in diameter. Dry silica sand, 75 percent or more of size 420-600 microns, is fed continuously onto the aggregate sample at a rate of 680-900 grams per minute. Following completion of the 500 revolutions, tray and aggregate are re-weighed and the percentage of aggregate loss is reported as the aggregate abrasion value (AAV). For aggregates to be used in first-class, heavily traveled pavements, it has been suggested by British researchers that this value should not exceed 10, and preferably be less for critical sections [Hawkes and Hosking, 1972].

3.8.1.2 Determination of Aggregate Impact Value (AIV)

This test gives a relative measure of the resistance of an aggregate to sudden shock. In this test, a cylindrical steel cup 4 in. (102 mm) in diameter, 2 in. (51 mm) deep and having walls not less than 1/4 in. (6.5 mm) thick is filled in three layers with aggregate particles passing the 1/2-in. (12.7 mm) sieve and retained on the 3/8-in. (9.5 mm) sieve. Both cup and aggregates are weighed. A standard hammer weighing about 30 lb (13.6 kg) is dropped onto the aggregates 15 times from a height of 15 in. (38 cm). The aggregates are then sieved on a No. 7 (2.40 mm) sieve and the ratio of the portion passing the sieve to the initial percent sample weight is reported as the AIV. The smaller this value is, the better the aggregate will resist impact forces. British specifications require that this value not exceed 30, and for surface aggregate, it is recommended that it not exceed 20 [Hawkes and Hosking, 1972].

3.8.1.3 Determination of Aggregate Crushing Value (ACV)

This test gives a relative measure of the resistance of an aggregate to crushing under a compressive load. In this test, a rigid, open-ended steel cylinder of nominal 6 in. (15 cm) internal diameter with a plunger and a base plate is filled to a depth of 4 in. (10 cm) with compacted particles passing the 1/2-in. (12.7 mm) sieve and retained on the 3/8-in.

(9.5 mm) sieve. The preweighed cylinder and sample are weighed and then the aggregates are subjected to a uniform compressive load of 40 tons (36.4 metric tons or 36.4×10^3 kg). After the load is released, the aggregates are sieved on the No. 7 (2.40 mm) sieve and the fraction passing the sieve is reported as percent of the original sample weight. An ACV$\leq$30 is the maximum allowed by British standards. Obviously, the smaller this value is, the greater will be the expected resistance of the aggregate to crushing.

3.9 A Suggested Aggregate Evaluation Procedure

The following procedure is suggested for evaluating aggregates without a performance history which are intended for use in pavement surfaces.

1. (a) Find the Los Angeles Abrasion loss for naturally-occurring normal-weight aggregates (sp. gr. 2.0 to 3.5). L.A. loss should not exceed 40 percent and should preferably be much less (25 percent or less) for aggregates to be used in critical or special pavement sections, or

1. (b) Find the Aggregate Abrasion Value (AAV), the Aggregate Impact Value (AIV), and the Aggregate Crushing Value (ACV), using British procedures (BS 812-75) or similar procedures when available, particularly if the aggregate does not lend itself to the Los Angeles test, as in the case of some synthetic aggregates. For first-class surfaces, the values of AAV, AIV, and ACV should not exceed 10, 20 and 20 respectively. In all three cases, lower values imply better performance. Target values not exceeding half of the allowable values are recommended for very high-performance surface aggregates.

2. Perform a laboratory polishing test that has been correlated with aggregate performance in service, e.g., British Wheel Test or any other polishing test. It is generally recommended that PSV or similar values should be no less than 60 and preferably 70 or higher for critical and special sections [Salt 1976].

3. Perform a petrographic analysis in addition to other routine weathering-related tests, and determine the susceptibility of constituent minerals to known or suspected chemical reactions and detrimental physical influences (e.g., wetting and drying, freezing and thawing). Also, the porosity of the aggregate should be determined. Research [Hosking 1974] has indicated that porosity in the range of 25 to 35 percent is recommended for an optimum combination of PSV and AAV. Porosity is not to be confused with water absorption. Normally, aggregates which have low absorption are more desirable than those that have high absorption.

4. Perform tests to check aggregate susceptibility to weathering, especially if a need for such tests is indicated by the petrographic analysis. Freeze-thaw tests and those that may indicate suspected chemical reactions should receive priority consideration in checking for weathering effects.

Correlations between laboratory tests and service performance are of utmost importance where available and should be used as guidelines in determining expected performance of new aggregates that have no performance history. It may be appropriate to mention here, however, that correlations between laboratory polishing of aggregates and skid resistance measurements between laboratory polishing of aggregates and skid resistance measurements on pavements incorporating the same aggregates have not been completely satisfactory. One important reason for the lack of satisfactory correlation is that skid resistance measurements in the field are made at a speed of 40 mph (64 km/h) that takes account of pavement surface macrotexture and probably other factors of surface characteristics, whereas laboratory measurements are made at low speeds (about 7 mph or 12 km/h) which reflect primarily microtexture characteristics and are hardly affected by the macrotexture. Nevertheless, several attempts in this area have shown that, in a general way, laboratory polishing procedures tend to rank the aggregates according to their level of skid-resistance performance when they are incorporated in pavement surfaces [Szatkowski and Hosking 1972, Gramling and Hopkins 1974, Mullen 1974, Dahir, Meyer and Hegmon, 1976].

4.0 CRITERIA FOR USE OF OPTIMAL PAVEMENT SURFACES

4.1 Introduction

Criteria for optimal pavement surfaces vary according to different situations of driver and environmental demands, particularly in some critical urban sections. Obviously, in all situations it is desirable to construct the most economical surface that will last the life-span expected of it with the least need for maintenance, and will provide the safest and most comfortable built-in performance--i.e., a surface that will be durable and, when wet, will present minimum or no hazard through skidding or hydroplaning potential under normal usage. It is also desirable that the same surface meet other criteria--least noise, least tire wear, minimum energy consumption through minimum rolling resistance,and no glare or other detrimental color effects. Satisfactory fulfillment of some of these factors, such as those associated with rolling resistance or glare problems may be compatible with the attainment of the primary requirements of wear and skid resistance, and may present little or no problem. Other factors may not be attainable to a high degree of satisfaction unless the primary considerations of durability and safety are compromised, or unless other measures are taken, such as restricting speeds. There are situations where these factors, normally considered secondary, may become primary factors. Examples include situations where roads or streets are close to amphitheaters, hospitals, schools, and similar institutions. Needless to say, the use of studded tires complicates problems of noise and rolling as well as wear and the hydroplaning potential associated with worn wheel tracks, but that is a subject that involves special problems and must be considered separately.

Characteristics of aggregates and textures of surfaces which will provide and retain wear resistance and skid resistance have been discussed. However, different situations require the use of different aggregates, designs, and textures which will result in the most economical and most prudent application of materials and procedures. Employment of optimal pavement surfacing systems to meet different conditions and requirements can best be delineated if factors other than wear resistance and skid resistance are discussed in some detail. These factors are discussed in separate sections under the following titles:

1. Aggregate and Surface Properties Vis-a-Vis Highway Noise
2. Tire Wear
3. Rolling Resistance and Fuel Consumption
4. Glare and Light Reflection
5. Other Aggregate and Pavement Characteristics.

4.2 Aggregate and Surface Properties Vis-à-Vis Highway Noise

In this section, we have considered several aspects of the questions of highway noise and of the relation of aggregate selection to it. As a beginning, we have sought to place the topic in perspective with regard to the overall traffic noise problem. We asked what level of traffic noise is acceptable in the neighborhood of a highway, and what is the degree to which actual traffic noise exceeds or falls below the acceptable levels. Answers to these questions define the seriousness of the overall highway noise abatement problem. Next, we focused on the role played by tire-pavement interaction noise in the total traffic noise nuisance. The answer to this question influences the weighting which the noise factor should receive in pavement design and in aggregate selection. We found, as will be discussed, that neither in highway noise design guide development nor in the FHWA standards, is the use of special pavement texturing advocated as a noise abatement measure. We found, also, however, that the main reasons for discounting pavement design as a viable noise abatement measure are now being diminished. It is our opinion that the payoffs for quiet texture should soon increase considerably.

Following the resolution of these general considerations, attention was directed to an examination of the specifics of tire pavement interaction, and an inquiry was made concerning the availability of mechanism models to account for the physics of the noise generating processes. We found the state of mechanisms modeling quite unsatisfactory, and have discussed the main deficiencies to some extent. We then reviewed some recent research concerning correlations between texture parameters and noise (correlations which satisfactory mechanism models should be able to explain.) Having determined that the present state of mechanism models does not provide answers to questions of aggregate selection, we turned, next, to an examination of empirical observations. Here we have been able to find some qualitative and rather tentative guidelines for aggregate grading formulations conducive to quieter pavements. Finally, in view of the incomplete understanding of these matters prevailing today, we have considered some recommendations for future research.

4.2.1 How Much Noise is Permissible

On April 23, 1976, and in continued response to requirements of the Federal Aid Highway Act of 1970, the FHWA published design noise levels applicable to various land uses adjacent to highways [Deputy Administrator FHWA 1976]. The design levels are listed in Table 4.1. A technical

Table 4.1. Design Noise Level/Activity Relationship

Activity Category	Design Noise Levels-dBA[1] $L_{eq}(h)$	$L_{10}(h)$	
A[2,3]	57	60	Tracts of land in which serenity and quiet are of extraordinary significance and serve an important public need and where the preservation of those qualities is essential if the area is to continue to serve its intended purpose. Such areas could include amphitheaters, particular parks or portions of parks, open spaces, or historic districts which are dedicated or recognized by appropriate local officials for activities requiring special qualities of serenity and quiet.
B[2,3]	67	70	Picnic areas, recreation areas, playgrounds, active sports areas, and parks which are not included in category A and residences, motels, hotels, public meeting rooms, schools, churches, libraries, and hospitals.
C[3]	72	75	Developed lands, properties or activities not included in categories A or B above.
D			Undeveloped lands.
E[4]	52	55	Residences, motels, hotels, public meeting rooms, schools, churches, libraries, hospitals, and auditoriums.

1. Either L_{10} or L_{eq} (but not both) design noise levels may be used on a project.
2. Parks in categories A and B include all such lands (public or private) which are actually used as parks as well as those public lands officially set aside or designated by a governmental agency as parks on the date of public knowledge of the proposed highway project.
3. Exterior.
4. Interior.

discussion of the history leading to publication of Table 4.1 has been reported by Rupert [1973].

The levels indicated in the table represent, it is stated, "a balancing of that which may be desirable and that which may be achievable." It is likely, however, that these levels will be attained in many cases only with considerable effort. For a perspective on the levels of Table 4.1, we note that in the presence of interfering traffic noise having an A-weighted sound level of 70 dB, outdoor activities requiring speech comprehension are considerably impacted [Miller 1971]. An L_{10} level of 70 dB, determined over a one-hour period, means that this level or higher would apply for 10 percent of that hour.

4.2.2 Observed Traffic Noise Levels Compared to FHWA Standards

Actual traffic noise levels are a direct function of traffic volume, terrain features, speed, kinds of vehicles (light, medium, or heavy trucks, or automobiles), and vehicle mix. For worst case combinations of these factors, and for unfortunate land use choice, the FHWA regulations can be substantially exceeded by today's highway traffic. For example, in the absence of man-made or natural barriers, traffic noise from a six-lane highway with 1200 vehicles/hour in each lane, 10 percent trucks, and an average speed of 50-60 mph (80-96 km/h) is said to exceed 71 dBA, fifty percent of the time, at a distance of 300 ft (98.3 m) from the near edge of the nearest lane [Serendipity 1971]. Since the L_{10} level is somewhat higher, any residential area within this distance would clearly be impacted in excess of the standard. By actual measurement, Galloway found noontime L_{10} levels exceeding 80 dBA in one particular roadside residential area [Galloway 1969]. This was the "worst case" finding in his investigation, though it is unlikely to be unique.

Properties adjoining the highway in this case were at grade level, and there were no intervening natural or man-made barriers. In most of the other measurement locations reported by Galloway in this study, the measured levels were within or much closer to present FHWA standards. More extensive measurements have been reported in years subsequent to the Galloway study [Kugler and Piersol, 1973; Rupert, 1973]. These support roughly the same conclusion; worst case data show noise impacts beyond FHWA standards.

In the preparation of the FHWA standards, it was recognized that worst case sites such as the above exist and that for these there are often overriding considerations opposed to amelioration. Even lower levels had been considered but the effect was judged such that accomplishment would have been ". . . beyond the reasonable capability of highway agencies to meet with highway measures alone," [Rupert, 1973].

4.2.3 Contribution of Tire-Pavement Interaction Noise to Total Traffic Noise

Traffic noise is a summation of many component sources of which tire-pavement interaction noise is the only one known to be dependent on pavement texture. It is important in the weighting of noise considerations in aggregate selection criteria, therefore, to know the degree to which the tire-pavement noise component participates in or dominates the total A-weighted sound level produced by traffic flow. The answer will depend, of course, on the traffic conditions, particularly on the speed of the traffic stream and on the mix between large trucks and passenger cars.

4.2.3.1 Single Vehicle Noise Data and Traffic Stream Noise Estimates

A considerable number of experiments have been performed on aspects of tire pavement interaction noise and on overall vehicle noise for single vehicles. A sampling of the results of such experiments is portrayed here in Figures 4.1 through 4.3. The remainder of this section seeks to clarify the portrayal.

Tire pavement interaction noise data are presented for the maximum legal vehicle speed of 55 mph (88.5 km/h) since at this speed the effect of pavement texture on the total noise impact is larger than at lower speeds. Moreover, it has been appropriate to refer passenger car pass-by noise data to a monitoring microphone distance of 25 ft (7.6 m), while truck pass-by noise data are presented for a 50-ft (15.2 m) microphone distance. These distances are the ones most commonly used for tire noise measurements for the two vehicle categories. The shorter measuring distance used for passenger car tire noise has permitted improved signal to ambient ratios to be attained.

Known extrapolation rules for referring data from one distance to another are subject to error, and though distance extrapolation was required in one instance, it has been avoided as much as possible. (In the one instance, a 5 dB increase was made in order to refer 50 foot (15.3 m) passenger car measurements to a 25 foot (8 m) reference.)

Some experimenters also reported data for speeds somewhat different from 55 mph (88.5 km/h). In such cases, speed extrapolation was also necessary. The adjustment was then made in the amount of 40 log (S/55), where S was the measured pass-by speed. (The largest speed extrapolation required was for data recorded at 45 mph(72 km/h) and errors from speed extrapolation are presumed to be small.)

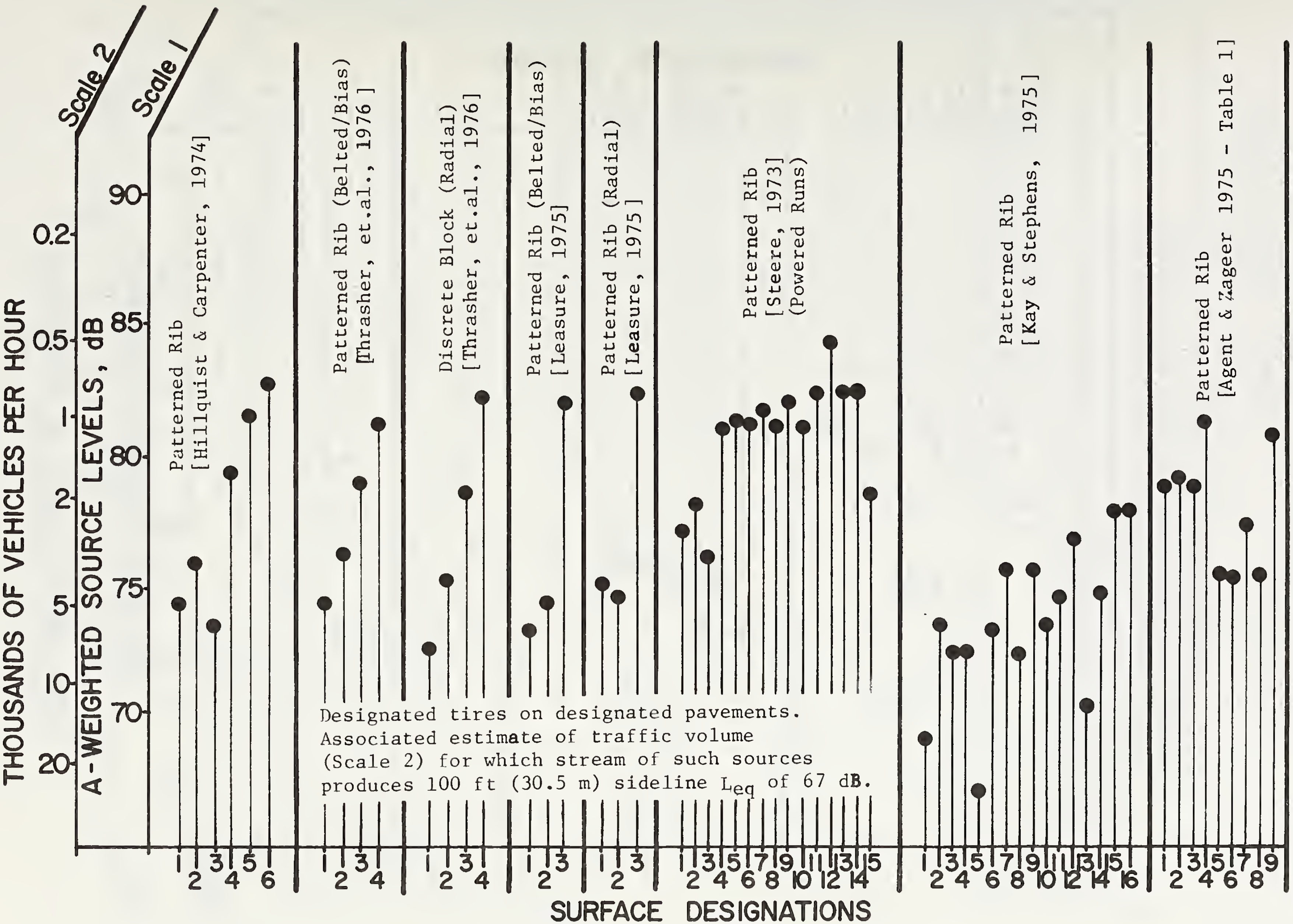


Figure 4.1. Source Levels (Scale 1) @ 25 ft (7.6 m) and 55 mph (88.5 km/h)for Passenger Cars

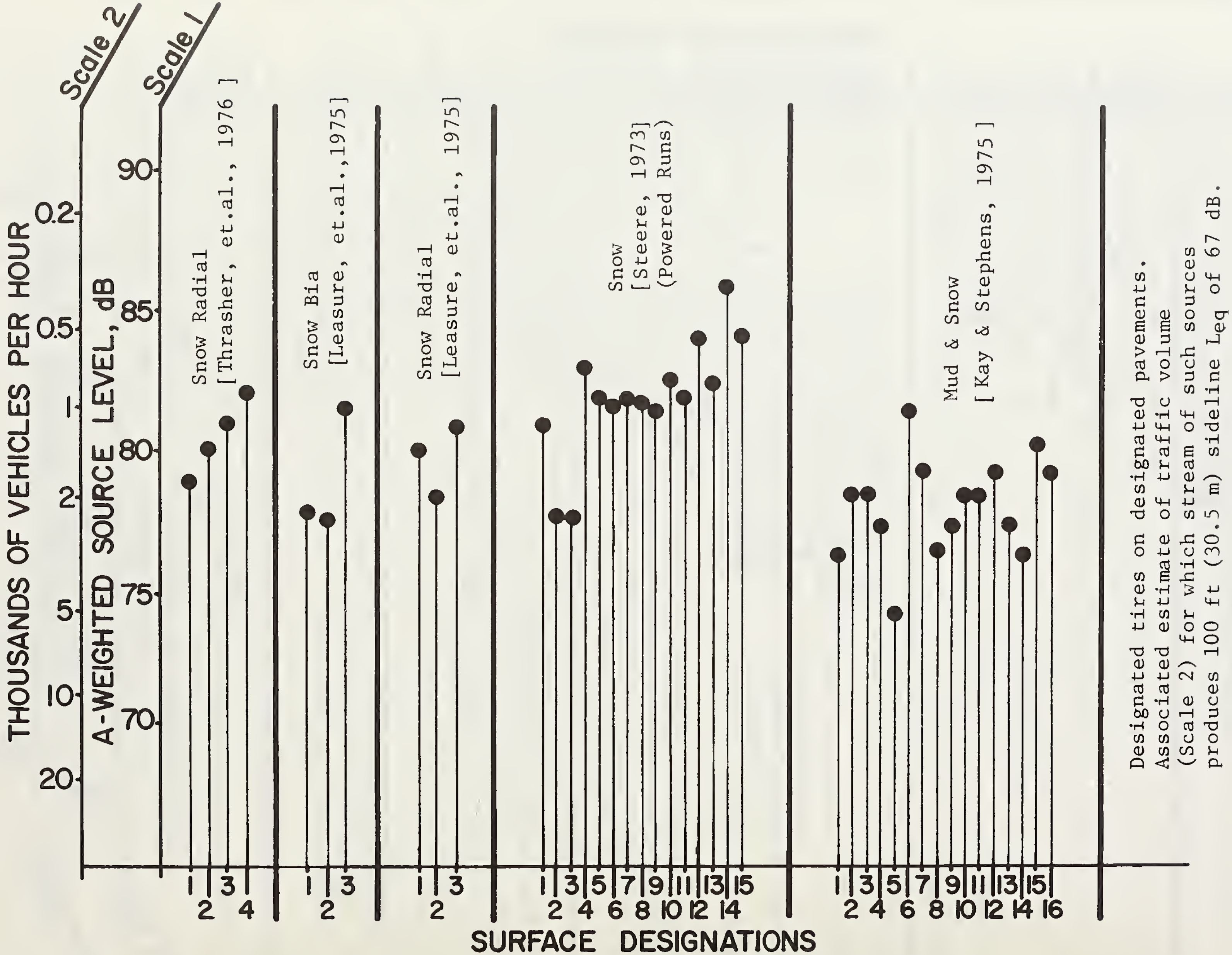


Figure 4.2. Source Levels (Scale 1) at 25 ft (7.6 m) and 55 mph(88.5 km/h) for Passenger Cars

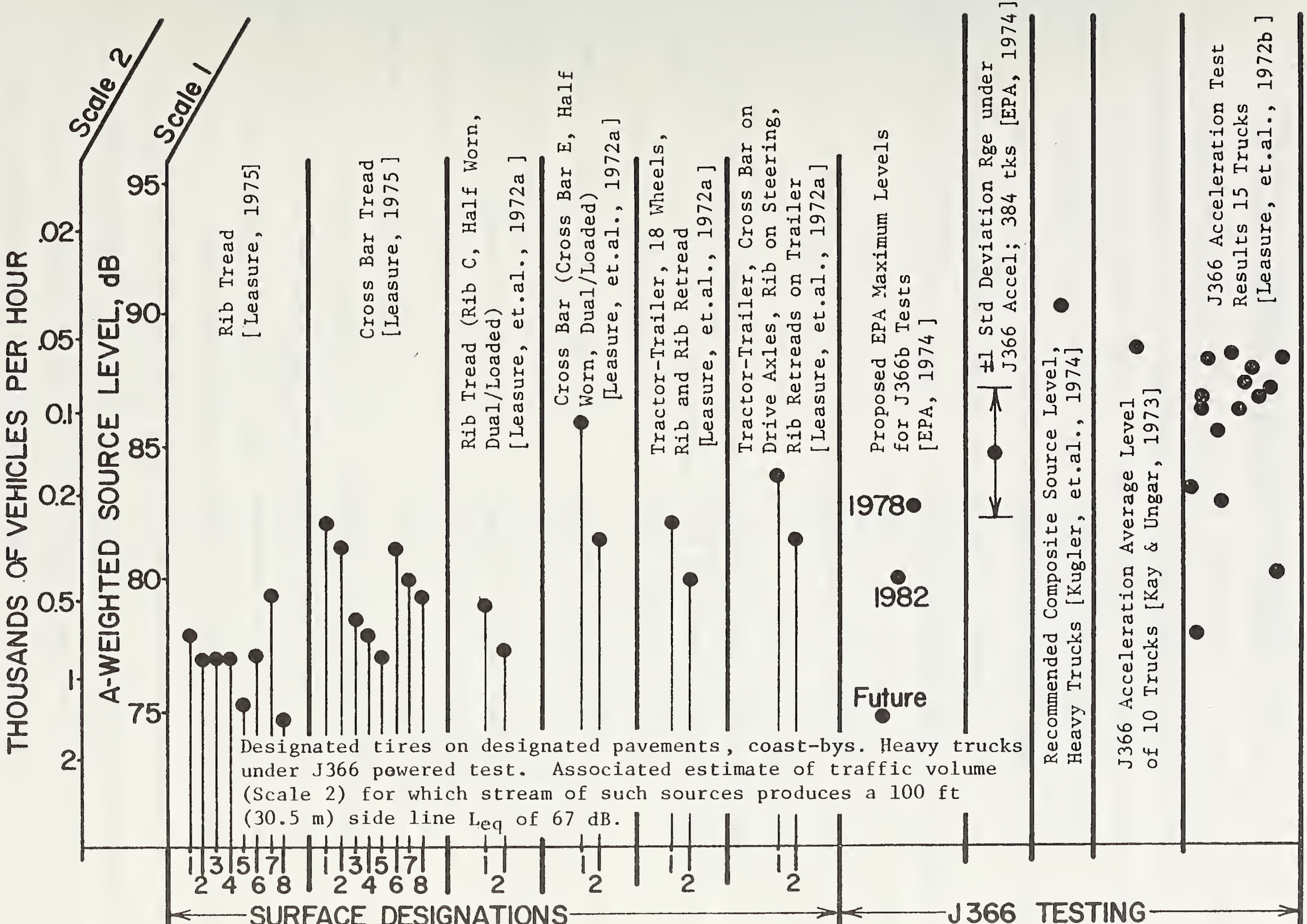


Figure 4.3. Source Levels (Scale 1) at 50 ft (15.3 m) and 55 mph(kk.5 km/h) for Heavy Truck Coast-bys

The investigators' findings are presented in separate columns in Figures 4.1 through 4.3. Where tire pavement interaction noise findings are reported, they are further separated according to tire tread type and plotted against designated pavement numbers (In some cases tire carcass construction information was available as well.) The latter numbers are, in each case, simply the order of presentation of the different surfaces in the noted original document. The surface texture descriptions, as taken from those of the experimenters, are listed in Table 4.2. In a few cases, aggregate gradation specifications for the pavements were given. These cases are noted in Table 4.2, and the gradation curves are plotted in Figures 4.4 through 4.6.

For passenger cars, no data under acceleration conditions are presented in the figures. An SAE J986a [SAE 1967] acceleration test level of 80 dB (at 50 ft or 16.4 m) is typical [Rentz and Pope 1974], but because of variety in muffler designs, passenger-car acceleration noise levels vary widely. (Under cruising conditions, passenger-car tire-pavement interaction noise may be significant even at speeds as low as 35-40 mph (56-72 km/h) [Leasure et al 1975].

It has been useful to portray SAE J366b [SAE 1969] acceleration noise data for heavy trucks. Such data, though representing some overestimation of engine-exhaust related noise under typical highway conditions, are thought to be close to total heavy truck noise, cruising at 55 mph (88.5 km/h), for trucks with crossbar treaded tires on the drive axles. Considering the tendency in heavy truck operation to run at approximately constant RPM at all forward speeds, the engine-exhaust component of total truck noise while cruising is not strongly speed dependent. Kaye and Ungar [1973], in work done for the DOT Quiet Truck Program, show a nearly constant engine-exhaust noise component for their datum vehicle but a rising, speed-dependent, tire-pavement noise

Table 4.2. Investigators' Surface Descriptions for Surfaces of Figures 4.1, 4.2, and 4.3[(1)]

Investigators	Surface No.	Surface Description
[Thrasher et al. 1976]	1	"Smooth Blacktop (low texture)"
	2	"Moderately smooth PCC" (Ohio Rt. 21)
	3	Worn track in PCC, exposed aggregate
	4	Cross brushed PCC
[Hillquist and Carpenter, 1974]	1	Smooth: sheet asphalt covered with several layers of highway marking paint
	2	Smooth: sheet asphalt with 3/8" max. aggregate randomly exposed
	3	Mild: asphalt surface - very homogeneous coating of 3/8" aggregate
	4	Rough: triple course; 1" maximum aggregate followed by 1/2" aggregate followed by 3/8" aggregate
	5	Rough: double course; 1/2" maximum aggregate
	6	Rough: single course treatment 3/4" aggregate
[Leasure et al. 1975]	1	Portland Cement "C" finish concrete, canvas belt smoothed, and/or "D" finish, burlap smoothed
	2	"B" surface course bituminous concrete (textured asphalt)
	3	Continuous poured reinforced concrete (unopened section of I-81 near Carlisle, PA)
[Steere 1973]	1-13 incl	Asphalt bound pavements and surface courses (see Figure 4.4)
	14	Old PCC (exposed aggregate) part of I-70 NE of Denver
	15	New PCC, medium texture using
[Kay and Stephens 1975]	1	New open graded asphaltic friction course
	2	New open graded asphaltic friction course

(1) 1" = 25.4 mm

Table 4.2. Investigators' Surface Description for Surfaces of Figures 4.1, 4.2, and 4.3 (Continued)(1)

Investigators	Surface No.	Surface Description
[Kay and Stephens 1975]	3	New open graded asphaltic friction course
	4	New open graded asphaltic friction course
	5	4-year-old open graded asphaltic friction course
	6	4-year-old dense graded asphalt concrete
	7	5-year-old dense graded asphalt concrete
	8	1-month-old dense graded asphalt concrete
	9	New dense graded asphalt concrete
	10	PCC broomed finish 5 years old
	11	PCC broomed finish 2 years old
	12	PCC broomed finish, new
	13	PCC broomed finish, new
	14	3/8" chip seal 3 months old
	15	3/8" chip seal 8 years old
	16	5/8" chip seal 2 years old
[Agent and Zegeer 1975]	1	Five Type A Bituminous concrete sites Figure 4.5 #1
	2	New Type A-modified Bituminous concrete sites Figure 4.5 #2
	3	Old Type A-modified Bituminous concrete sites Figure 4.5 #3
	4	PCC nine tests at seven sites
	5	Transverse plastically grooved PCC
	6	Sand asphalt, 13 sites 7 tests
	7	Kentucky Rock Asphalt, 5 tests Figure 4.5 #7
	8	Open graded plant mix surface Figure 4.5 #8
	9	Chip seals Figure 4.5 #9
	10	Rough, cracked, and bumpy

(1) 1" = 25.4 mm

Table 4.2. Investigators' Surface Description for Surfaces of Figures 4.1, 4.2, and 4.3 (Continued)(1)

Investigators	Surface No.	Surface Description
[Kilmer et al. 1975]	1	PCC, belt drag, finish, clean
(also see Figure 4.6)	2	"Jennite" asphalt concrete, clean and polish
	3	Limestone hot mix asphaltic concrete with Terrazo finish, polished and cleaned
	4	Crushed gravel, hot mix asphaltic concrete, washed and polished
	5	Rounded gravel hot mix asphaltic concrete, washed and polished
	6	Rounded gravel, asphalt chip seal
	7	Lightweight aggregate asphalt chip seal (synthetic aggregate, fired clay)
	8	Lightweight aggregate hot mix asphaltic concrete, synthetic (fired clay) aggregate
[Leasure et al. 1972]	1	PCC smooth (same surface as [Leasure et al. 1975] surface #1)
	2	Asphalt concrete (same surface as [Leasure et al. 1975] surface #2)

(1) 1" = 25.4 mm

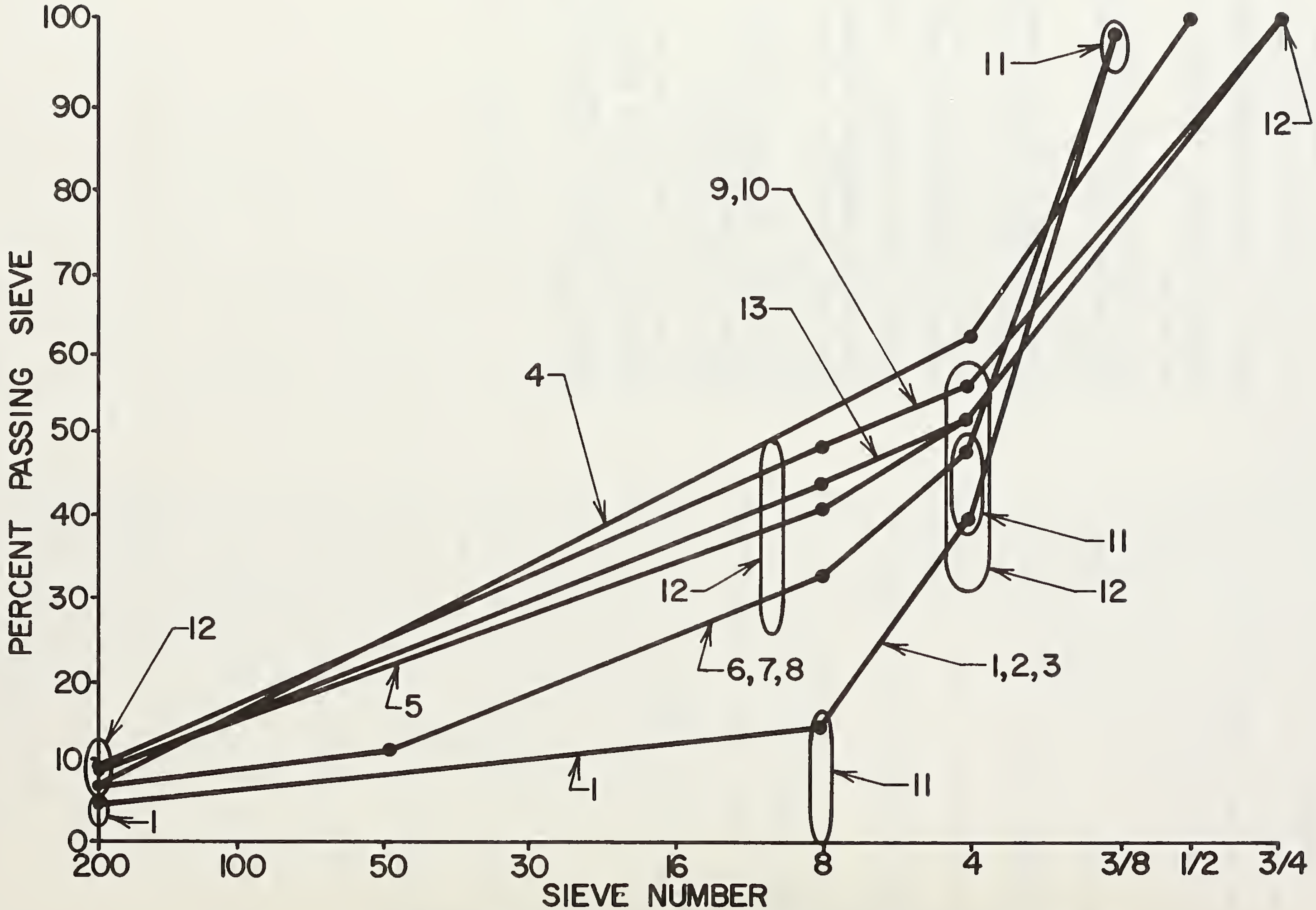


Figure 4.4. Pavement Aggregate Gradation for Several Pavements on Which Tire-Pavement Interaction Noise Tests were Run [Steere 1973]

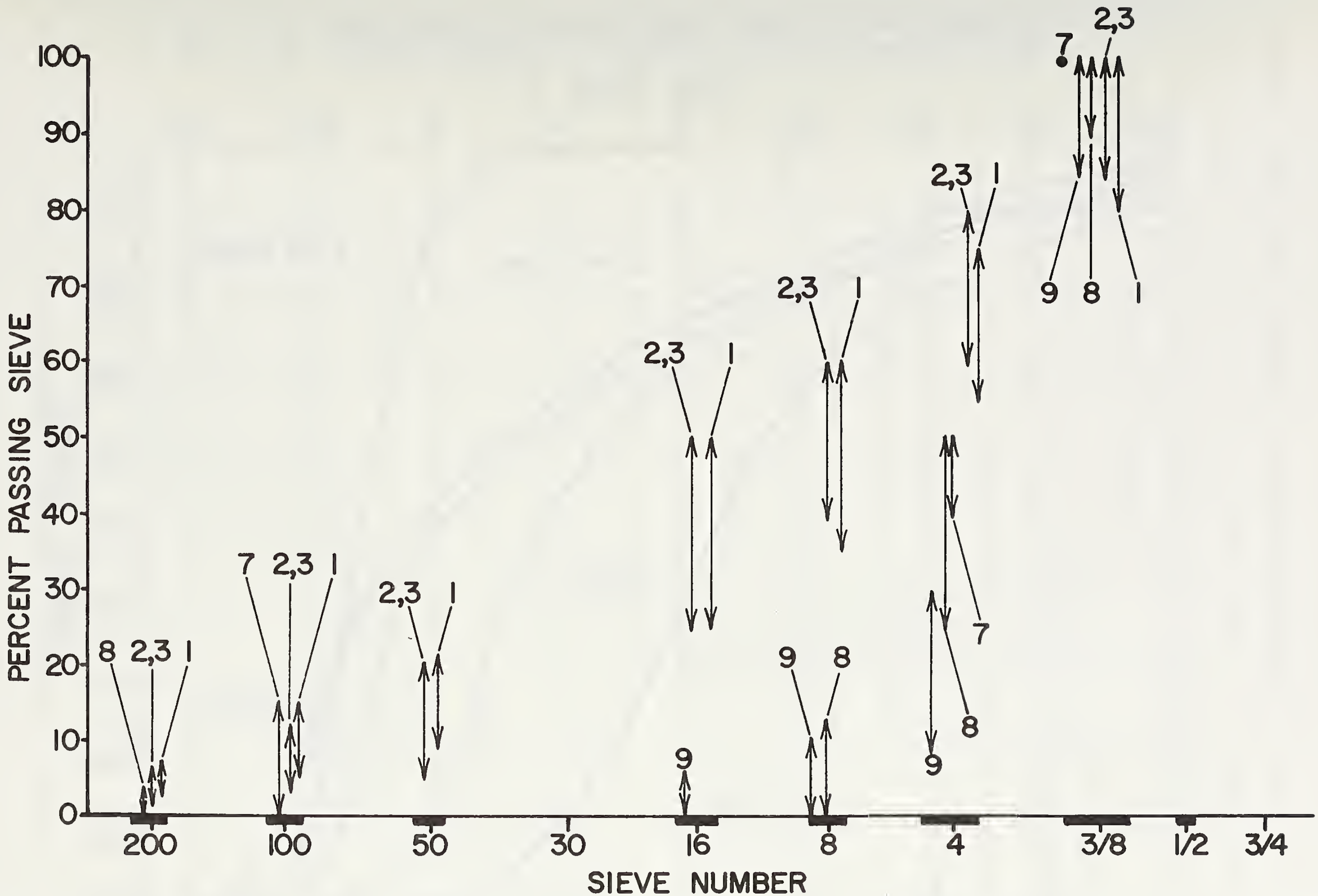


Figure 4.5. Pavement Aggregate Gradation for Several Pavements on Which Tire-Pavement Interaction Noise Tests were Run [Agent & Zegeer 1975]

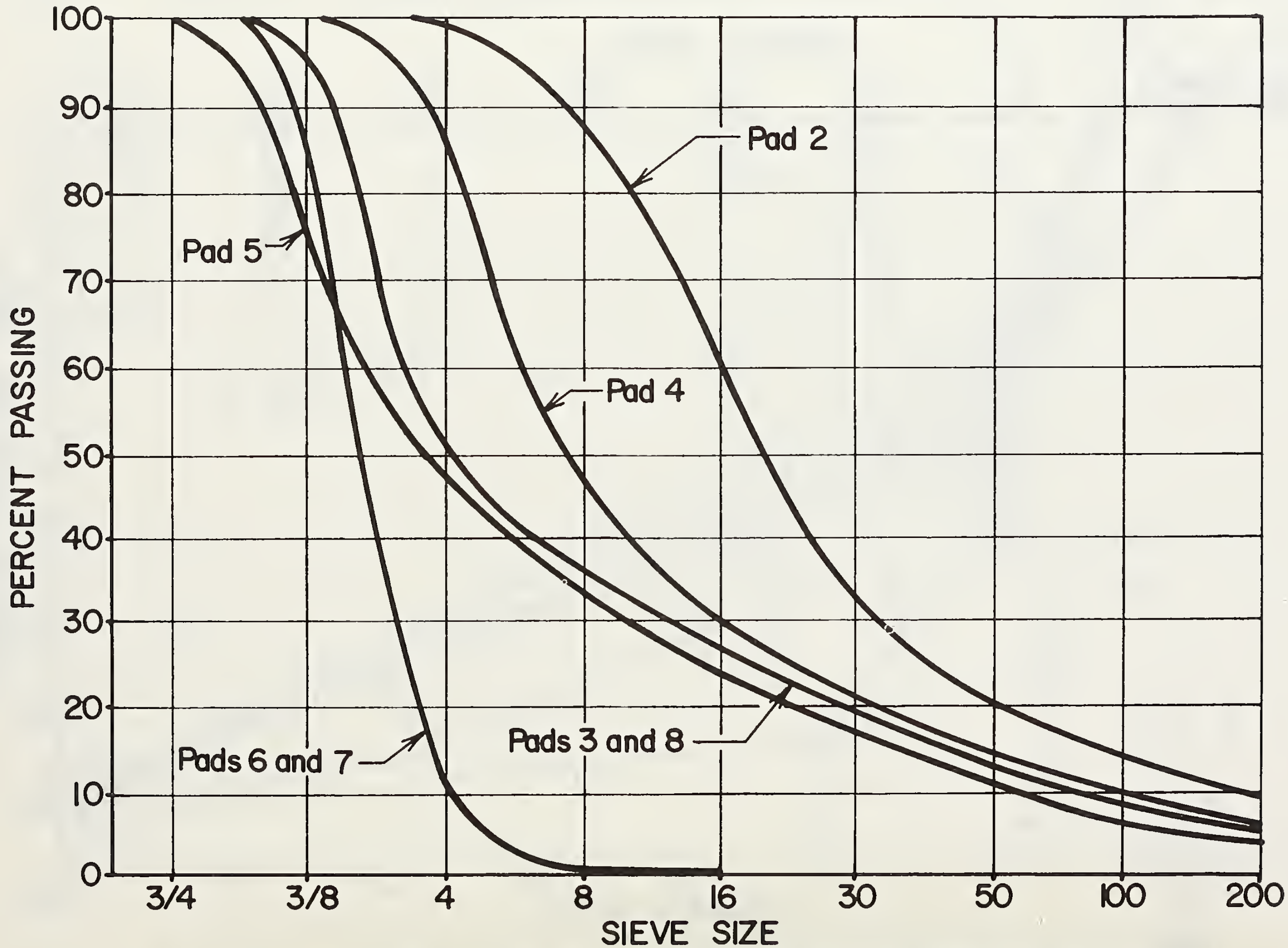


Figure 4.6. Pavement Aggregate Gradation for Several Pavements on Which Tire-Pavement Interaction Noise Tests were Run [Kilmer et al. 1975]

component. The latter, moderately extrapolated, equals the former at a speed of about 55 mph (88.5 km/h) at which speed the total A-weighted noise would be approximately 86 dB at 50 feet (15.3 m). This level is well within the spread of J366b data shown in Figure 4.3.

That the J366b acceleration data for heavy trucks is close to total heavy truck noise while cruising is further suggested by the 90 dB A-weighted speed independent source level recommended by Bolt, Beranek, and Newman for use in predicting heavy truck contributions to traffic stream noise [Kugler et al. 1974]. This 90 dB point has been plotted in Figure 4.3. Year 1978 and year 1982 EPA regulatory levels for truck noise, as well as the mean and the one standard deviation limits of an EPA-run J366 acceleration test sample, are also shown in Figure 4.3. The 1982 regulation (and an anticipated and considerably lower 1985 regulation) is based, in great part, on demonstrated attainments of the aforementioned quiet truck program.

In order to show the significance of individual vehicle source levels in Figures 4.1 through 4.3, a second ordinate scale has been added to each figure. Associated with each source level as read from Scale 1 is a traffic stream volume on Scale 2. This volume is the number of such sources per hour passing a fixed microphone point (100 feet (30.5 m) from the (single) line of sources) which is just sufficient to raise the L_{eq} level of traffic stream noise to the allowed FHWA maximum for Type B land uses (67 dB, A-scale).

The model used in aligning Scale 2, i.e., in relating source emission level to traffic stream level, is drawn from a recent design guide recommendation to NCHRP [Kugler et al. 1974]. It requires, as input information, the source level at 50 ft (15.3 m), and it assumes a 10 log $\frac{D}{50}$ reduction in traffic stream noise beyond that distance. (Additional corrections for terrain loss, and the like, are not used here in computing the 100 foot (30.5 m) sideline performance). Errors inherent in the model are, therefore, augmented for passenger car data by any extrapolation error in referring single source levels to 50 ft (15.3 m). Conclusions arrived at using Scale 2, especially conclusions related to passenger cars, must be considered as tentative, therefore, unless they are insensitive to one or two dB changes in apparent source levels or are based on comparisons where errors would be in the same direction among items compared.

4.2.3.2 Under What Circumstances is Tire Pavement Noise Significant

An examination of the J-366 levels and related traffic volumes of Figure 4.3, taken in light of the discussion in the previous section which relates these levels to 55 mph (88.5km/h) cruising levels, indicates that, with few exceptions, a volume of 75-150 heavy trucks per hour is sufficient, alone, to produce 100 foot (30.5 m) L_{eq} levels equal to or above the FHWA Type B land use maximum.

Figures 4.1 and 4.2 show that in most cases more than ten times as many passenger cars per hour would be required to produce the same traffic stream level, even over the noisier textures or with snow tires. Over average textures, and without snow tires, the passenger car-truck cruising noise equivalent is clearly greater than ten cars to one truck). As long as there is no reduction in truck engine/exhaust related noise components, therefore, traffic streams with ten percent or higher proportions of trucks will be insensitive to differences in passenger car noise. Moreover, the change from cross-bar to rib-tread truck tires, whether to the somewhat quieter radials or to bias ply types, will produce little change in overall 55 mph exterior truck noise unless accompanied by reductions in engine/exhaust related components.

These considerations lead to a noise ranking of worst case 1977 traffic scenarios in the following order:

1. Traffic with high percentages of heavy trucks, accelerating at full throttle
2. High-speed cruising traffic with high percentages of heavy trucks (a high percentage of trucks in 1977 are fitted with cross bar tires)
3. Moderate-speed cruising traffic with high truck percentages
4. Accelerating high volume traffic, even with moderate to low heavy-truck percentages, and
5. High-speed cruising traffic with high volume flow and with low to zero percentages of trucks.

Automobile tire noise is not a dominant factor in any of the first four cases, considering the noise levels of today's trucks and automobiles. (Future noise abatement of truck engine sources will change this conclusion, as will be discussed later.) Since engine related noise is not a function of pavement texture, and since cross-bar-tread tire noise is not strongly dependent on the texture over the available range of truck tires with cross-bar treads [see also Close and Wesler 1975], it follows that pavement texture modifications will have little impact in any of the first four cases.

In the fifth case, tire-pavement interaction noise is the dominant source, and since automobiles, which have a predominance of rib-tread tires, make up a majority of this traffic stream, pavement-texture design will affect the noise impact. (Note the wide variation of noise with texture reported by several of the investigators in Figure 4.1.) When high speed (55 mh, or 88 km/h) freeway sites have automobile traffic in excess of 1500 vehicles/hour, the dominating tire-pavement noise will, for some pavements, be more than is allowed by FHWA standards for Type-B land use at distances beyond 100 feet from the near traffic lane. Only on these and more densely traveled high-speed freeways would automobile traffic alone present a noise high enough to require mandated action. Only in cases where, in addition, truck percentages are low would one pavement appear to be significantly quieter than another in today's traffic.

4.2.4 Traffic Noise Prediction Models

Some of the statements made in the last two sections stem from research on highway-noise prediction models. Some further illumination of the present need for noise considerations in aggregate selection comes from a review of the state of these efforts with regard to pavement texture effects.

Under sponsorship of the state highway agencies (through the NCHRP), and with support by, and close coordination with, FHWA, two approved design guides for predicting traffic noise have been prepared [Gordon et al. 1971, Wesler 1972]. These design guides include manual and/or computerized formulations for predicting the A-weighted sound level of traffic noise at any specified location, given values for traffic volume, speed, terrain features, truck percentages, and the like. The two design guides are accepted options in the preparation of impact statements for Federally-aided highway construction projects. The factor most relevant to the present study, pavement texture, is among the least successfully handled factors in the NCHRP guide, and it is not included in the TSC-development model. The difficulties have been twofold: first, quantitative measures of pavement texture were just becoming available, and were primarily related to skid resistance, at the time the design guides were being developed. Second, the role of pavement texture, however characterized, had not been factored into the available mechanism models for the tire-pavement interaction noise generation. In fact, no useful tire-noise mechanism models existed, a deficiency which continues even today. The crude pavement descriptions and the corresponding noise level adjustment of the 1971-issued NCHRP design guide are indicated in Table 4.3.

After several years of experience with the design guides, it had become apparent that a revised and improved version would be necessary, independent of pavement texture considerations. Therefore, under NCHRP project 3-7/3, a further guide development was initiated by Bolt, Beranek & Newman, Inc. [Kugler 1976, Anderson 1976]. A final report on this effort was completed in 1974 [Bolt, Beranek & Newman, Inc. 1974], and has now been released [NCHRP 1976]. It appears, however, that Table 4.3 remains the guideline for purposes of classifying pavement texture. Moreover, it appears that classifications other than "normal" are to be noted as unsatisfactory surfaces for ride comfort or traction performance. Thus, new and resurfaced pavements (for which predictions are required if the pavements are part of a Federally-aided project) would in all cases have to be regarded as "normal." Pavement texturing, if current design guide thinking prevails, will not be accepted as a means of bringing traffic noise levels within the land use standards in proposed plans for Federal-aid highway construction. It seems possible that this point may act as a deterrent to potential developers of means of quiet pavement texturing. At least in the short run, a case appears to have been made for a low weighting for the noise factor in the tradeoffs pertinent to optimum aggregate selection.

4.2.5 Future Payoffs for Quiet Texture Development

In the long run, it is believed, pavement texture effects on traffic noise will be reconsidered. Evidence continues to mount supporting the wide

Table 4.3. Classification of Road Surface as it Relates to Surface Influence on Vehicle Noise [1]

Surface Type	Description	Adjustment (dB)
Smooth	Very smooth, seal-coated asphalt pavement	-5
Normal	Moderately rough asphalt and concrete surface	0
Rough	Rough asphalt pavement with large voids 1/2-in. or larger in diameter, grooved concrete	+5

(1) 1 in. = 25.4 mm

variability of traffic noise data from site to site, attributable in part, perhaps, to pavement texture differences [Kugler 1976]. Major sources of traffic noise are coming under Federal control, e.g., trucks have been designated major noise sources and placed under control by the EPA, as indicated by the regulatory levels of Figure 4.3 [EPA 1976a]. DOT-sponsored truck noise research has indicated the feasibility of substantial source level reduction, particularly in the dominant exhaust noise component [Kaye and Ungar 1973, Shrader 1975, G. Williams et al. 1975]. Exhaust noise, the major contender besides tire-pavement noise for the composite source level of automobiles, is already the lower of the two sources under high speed cruise conditions, and muffler technology is available to reduce this component further. Tire noise research, as exemplified by the results of Figures 4.1-4.3 has shown that the payoffs in tread design are mainly those in switching from cross-bar tread to rib tread configurations. (The trend toward a radial rib design is now being seen in the trucking industry, largely because of the increased life obtained from this design.) With engine/exhaust noise components reduced, the known sensitivity of the soon-to-be-prevalent rib treads to pavement texture will remain to be exploited. All these considerations suggest to the investigator that the payoff for quiet pavement designs must increase in the future. For asphalt-based designs, in particular, the weighting of noise in aggregate selection criteria will then increase as well.

4.2.6 Tire Pavement Interaction Noise Models

In order to establish more closely the role of aggregate selection in the noise phenomena mentioned above, the literature has been perused for papers on the mechanisms of tire-pavement interaction noise. A number of alternative mechanisms have been proposed, though quantitative models have been either non-existent or overly simplified. One mechanism which includes texture effects is the so-called air pumping mechanism, first enunciated by Hayden [Hayden 1971]. Hayden presumed that the tire traps air both between tread elements and within cavities in the pavement texture, alternately compressing the trapped air and releasing it as new tread and pavement elements enter and leave the footprint area*. The texture-related parameters in Hayden's model are cavity volume, the degree of penetration of the tire into the cavity volume (i.e., the volume change of the entrapped air), and the number of cavities per second closed or opened by the moving footprint. Although each of these parameters is clearly a random variable over the pavement, Hayden's computation was for regularly spaced cavities and did not show the effects of random variation.

Another noise mechanism in which pavement texture plays a part is the tread vibration mechanism [Hillquist and Carpenter 1974]. Tread vibration levels have been estimated from accelerometer data, with accelerometers mounted inside the tire [Richards 1974] and have been observed directly by measurements on tread elements and sidewall vibrations over differing pavements and for tires with different amounts of wear [Reiter et al. 1974]. Some simplified computations of the energy stored, and at least partially released as noise, by tread "slap down" have been made by Archibald [1974] and Buchan [1974]. The Richards report discusses the effect of random texture in modulating the tread noise. Unfortunately, this vital aspect of the texture effect is not quantitatively set down, and the texture

*At the 1976 SAE Tire Noise Symposium, the meaning of air was called in question. In particular, the possibility of tread cavities acting as acoustic reactances responding to tread vibration was suggested as an alternative explanation to that of pure monopole piston pumping (in which the tire rubber acts merely as the wall of the pumping chamber).

parameters most important in the process are not stated. A beginning in this direction has recently been reported from Sweden [Nilsson 1976] where a one-dimensional model of the surface as a source of random mechanical interference with a treaded tire has been advanced. Again, however, an appropriate statistical description of the surface texture has not been introduced.

The third commonly mentioned mechanism for tire noise is Siddon's vortex shedding mechanism [Siddon 1972]. Siddon's postulated mechanism does not include a texture effect, however, and must be discarded or modified since it does not account for the observed noise differences generated by a given tire over a range of pavements.

In addition to an inability to account for texture statistics, all the current modeling efforts are deficient in several other respects. For example, while it is known that truck tire noise is directional, no quantitative explanation for the directionality has been advanced. Further, although some allusions have been made to pavement porosity and to the propagation loss of tire noise over porous surfaces, the subject has not been explored at any length. Recent measurements by National Bureau of Standards investigators on the acoustic impedances of pavement surfaces will be directly relevant [Mansbach and Holmer 1977], but await the development of the source model. Truck tires are generally paired and often run on dual axles. Although the circumferences of individual tires are not precisely the same, they are generally quite close, so that tread generated noise, while not in synchronism, may be expected to demonstrate some interaction. This phenomenon has not been modeled. Truck and trailer bodies come in many shapes, and the sideline noise produced by tire-pavement interaction may be sensitive to these varying reflecting surfaces. Tires on inside wheels are shadowed by tires closer to the monitoring microphone. This shadowing effect, which was observed in mix and match pairing experiments by National Bureau of Standards (NBS) investigators [Anon. 1970], has not been satisfactorily accounted for.

In brief, there is still controversy over the mechanism issue, and none of the alternative mechanism proposals have been taken far beyond the concept stage. Certainly, there is no adequate model of tire-pavement interaction noise today, and textural aspects of the modeling problem are not in hand.

4.2.7 The Role of Pavement Texture in Tire-Pavement Interaction Noise: Recent Observations

It has been noted above that the accepted mechanism models for tire-pavement interaction noise do not provide for the insertion of quantitative values of any pavement texture statistical parameter. Pavement texture statistical descriptions have been almost exclusively the result of concerns for improved skid-resistant surfaces. The common measures of the texture have been discussed elsewhere in this report, but it is appropriate here to indicate the few findings available on how some of the measures correlate with measured noise levels of tires rolling over variously textured pavements. It is fairly well agreed that pavement texture within the macrotexture wavelength scale is primarily responsible for tire-pavement noise at traffic speeds where such noise is significant (a 55 mph or 88.5 km/h pass-by of a smooth tire over a sinuous transverse-grooved texture of 1/2-inch or 12.7 mm wavelength would generate a tone at a 1936 Hertz fundamental frequency). Based on a fairly small sampling of surfaces, linear correlations between pass-by noise and "Void Area" have been reported [Fuller and Potts 1975]. Void areas were determined from six-inch (152-mm) profile traces of several Texas Transportation Institute sites. (These sites are the same as used by the NBS team--Kilmer et al. 1975.) Essentially no change in correlation was obtained through multiple regression, using as additional parameters the measurements determined from the sand patch test and the skid number gradient. Better linear correlation (.88) was found when the two very smooth (low drainage area) sites were excluded. Just as with the Hillquist and Carpenter data in Figure 4.1, pass-by noise levels on these smooth sites had actually exceeded those on sites with moderately high macrotextures. It is suggested that a different mechanism may be operative on smooth sites. The Fuller-Potts results, while of considerable interest to model makers, are specialized to the neutral ribbed ASTM tire tread. No such correlations have been found for commercial treads.

Any mechanism model which is complete will have to account for the above-mentioned nonlinear level change with drainage area or similar measure of texture. It is clear, for example, that a surface with no macrotexture-scale roughness will continue to interact with smooth tire rubber to generate noise, for smooth tires running against polished steel drums do so [Richards 1974]. Richards' explanation for this blank-tire-smooth-texture noise was "tread squirm" which, he speculates, is vibration in small, localized portions of the tread.

Fuller and Potts found essentially insignificant correlation between pass-by noise and skid number, between noise and skid number gradient, and between noise and the sand patch test measure of texture depth, as well as multiple regression coefficients among these variables. In different measures, the Fuller-Potts report and reported results of tests done at PTI [Veres et al. 1974] show rank ordering between tire-pavement noise levels and measures deduced from space frequency spectrum analysis of pavement texture. Fuller and Potts' data on pass-by A-weighted noise levels rank-correlated best with an integrated spectrum measure over the 100 cycle/

meter to 350 cycles/meter band of texture frequencies*. Veres' comparison of PTI data pertained to nearfield tire noise, which he was trying to use as a measure of texture. He found best ranking among texture spectra at PTI sites using a limited (1/3 octave) band of tire noise around 1600 Hertz. He, too, however, was using a treadless (commercial) tire. The presence of a tread alters ranking in a way not understood. No significant correlation between noise and any specific texture parameter has been found for commercial treaded tires.

Empirical associates of noise and texture measures are encouraging, but ultimately unsatisfying because of a lack of physical insight into the effect of texture randomness parameters on the noise generating mechanism. It is not clear whether some other texture statistic (such as the most probable wave number), or some combination of a few statistics (e.g., the first few central moments of the texture profile height distribution) might relate more closely to the generated noise. Moreover, the parameters intuitively best suited to the monopole air pumping model of the source mechanism (e.g., volume outflow test number) are probably not those most suited to the tread vibration model.

4.2.8 Considerations in Aggregate Selection

Despite the lack of completeness of the modeling and correlation findings above, some tentative inferences may be drawn as to the selection and use of aggregates for quieter pavements. For one thing, there is some evidence that the objectives of quieter pavements and of pavements with

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*Fuller and Potts' spectra on their Texas Transportation Institute sites do not agree with PTI spectra for these same sites. A private communication with Potts on these differences indicates that the spectra displayed in their report are not third octave band spectra, but are fixed bandwidth displays. As there may be additional discrepancies, only the relative ranking information has been given credence in this analysis.

ample skid resistance are not diametrically opposed. It may be noted from Figure 4.1 and Table 4.2, for example, that some of the new open graded asphaltic friction courses are among the quietest pavements reported, at least for passenger cars. The Kay and Stephens data for their sites 1 and 5 and the Steere data for his sites 1-3 are cases in point*.

From the point of view of monopole air pumping, an open graded texture is to be preferred, since pressurization of texture cavities is less likely. Such textures, moreover, are not associated with annoying tread frequency noise "tonals," as on the contrary, is smooth textured PCC, for example.

Several investigators have reported a nonlinear variation of noise level with (crude) texture measures. Hillquist and Carpenter's surfaces generally increase in macrotexture-scale roughness with increasing surface designation number, yet his reported passenger-car rib-tire noise levels (Figure 4.1) on these surfaces show a minimum at an intermediate texture. Fuller and Potts' FHWA contract report [Fuller and Potts 1975] notes a similar finding, as has been mentioned. The textures at the smooth end of the scales of these investigators, however, are such that skid-resistance properties are known to be poor. Furthermore, the noise behavior of such surfaces with snow tires and cross-bar tires is less favorable. The optimum noise performance for pavements is, thus, dependent on the nature of the tire tread. As the tendency toward radial tires develops, and assuming cross-bars are phased out to a considerable extent, it is expected that a sharper optimum will exist. Even so, the difference in tread pattern scales between passenger car tires and heavy truck tires suggests that the

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*The generally quieter trend in Kay and Stephens' data compared to the others is not explained and needs further corroboration. If real, the presence of a geographic factor, e.g., one yielding differences in aggregate properties, may be involved. In the meantime, it is probably safer to give more attention to relative data for each investigator rather than to make comparisons among investigators.

texture which is optimum for trucks may not be so for passenger cars. Then the optimum will depend on truck percentage.

Present findings suggest a further guideline. Steere's results in Figure 4.1 show slightly lower noise levels for surface numbers 1-3. All his other gradations had somewhat larger percentages of aggregate passing the number 8 sieve, except his number 11, which was considerably older. Some of the surfaces tested by Agent [Agent and Zegeer 1975] were also of open graded plant mix type, as indicated in Table 4.2. These Kentucky tests yielded qualitatively similar results to Steere's with regard to noise performance for new open graded plant mix surfaces, but it was found that, in Kentucky, the plant mix surfaces experienced severe wearing after about a million vehicle passes, whereafter texture was considerably modified and noise levels rose. (Colorado Type A plant mix sites 1-3 had not reached the million vehicle pass point at the time of Steere's tests.) Agent found better noise-versus-wear performances with Kentucky Quartz Rock Asphalt and Kentucky Quartz Sand Asphalt pavements, while noise levels at all stages of wear were only slightly greater than for the plant mix seals. The Rock Asphalt aggregate mix was given as:

100 percent passing 1/2" sieve, 40-50 percent passing No. 4 sieve, and 0-15 percent passing No. 100 sieve. (1/2" = 12.77 mm)

Agent and Zegeer's photograph of their "Kentucky Rock Asphalt" surface, site number 7, does not suggest the openness of, for example, his site number 8. It has been suggested [private communication from J. Rice] that the tolerance of the percent passing the number 4 sieve may have been broader than that stated.

The Kay-Stephen findings as to the improved noise performance of open graded asphaltic friction courses were the result of a three state test program conducted by the San Francisco regional office of FHWA; see also

W. Williams et al. [1975]. Again, it was the new open graded surfaces, still in the "rolling-in" stages which were quieter. This finding clearly requires further inquiry. Unfortunately, apart from the indirect evidence such as age and total vehicle passes, and mention of deterioration [Agent and Zegeer 1975], the variation of actual surface texture of test surface types with time has not been included in the investigators' reports.

4.2.9 Brief Note on PCC Noise

Although the present inquiry has been directed toward factors related to aggregate selection, it should be noted that textured Portland cement concrete surfaces are included in the sites tested by most of the investigators. Thrasher et al.'s site number 4, Leasure's Carlisle site (site number 3 of his 1975 findings), and Agent and Zegeer's site number 5 are anisotropically textured with pronounced transversely running markings (cross brush marks in Thrasher's and in Leasure's sites, transverse grooves made by combing the plastic concrete in Agent and Zegeer's site). Tires running over all three of these surfaces were exceptionally noisy. On the other hand, while the levels of noise generated by aggressive treads, particularly cross-bar treads (or the now illegal pocket retreads) remain high on smooth PCC, the less aggressive rib-tread tires become considerably quieter. The authors have not found comparable information with respect to longitudinal texturing. For passenger cars, worn PCC, as in Thrasher's surface #3, in which the aggregate is exposed, is considerably noisier than new, smooth PCC, as for his surface #2, except for cars fitted with snow tires which are somewhat less sensitive to texture changes.

4.2.10 Research Recommendations

The investigators believe that the following areas of research should be pursued to understand why tires are quieter on some pavements than on others, and to know how to design pavements (including aggregate selection) so as to reduce tire-pavement interaction noise without compromising other performance measures inordinately:

1. Examine a variety of pavement-texture statistical measures over a range of pavements and determine which are appropriate for ranking and/or predicting high speed coast-by tire noise. Relate these statistical measures to aggregate grading formulae for asphalt concrete pavements, and to brushing, dragging, raking, and other texturing processes for Portland cement concrete. Relate them, also, as to ability to estimate surface traction performance, or examine their interdependency with other statistics which do rank traction performance. Initially the scope of this task might be limited to such a range of surfaces as are readily accessible to a single investigating agency. For example, several organizations have sets of test strips of differing textures among their transportation research facilities. A three or four man-year level of effort is suggested as appropriate for the task. If the pay-off promise is clear, then a broader program aimed at sampling representative surfaces throughout the country would be in order.

2. Examine differences among rib tread, cross-bar tread, and blank tread tires in the response of local tread regions to small area impact. Determine the tread response dependency on such impacting parameters as the diameter of the area of contact at impact, the angularity of the impacting object, the velocity of the impacting object at the instant of contact, and the penetration depth prior to a release-type impact. Determine the extent to which tire carcass influences tire response to local high speed impacts (as by individual stones), thereby providing a quantitative description of the tire response (spatial and temporal) to a single, local impact, so that the composite response due to random impacting may be predicted (given appropriate texture statistics). This investigation should be within the scope of a two man-year effort.

3. Develop procedures for quantifying and conduct field investigations to quantify the differences in textural characterization tween open graded and dense graded pavements, between new and worn-open graded, plant-mix seal coats, and between such surfaces under different climatic conditions. This investigation should probably consist of two phases: first, the development of procedures by a specialist in this area, and, second, the use of these procedures for an examination of a representative sampling of sites. The latter sub-task might be suitable for regional offices of the FHWA, perhaps with specialist monitoring.

4. Examine the propagation of tire noise over porous textures and determine measures of texture appropriate to a propagation model.

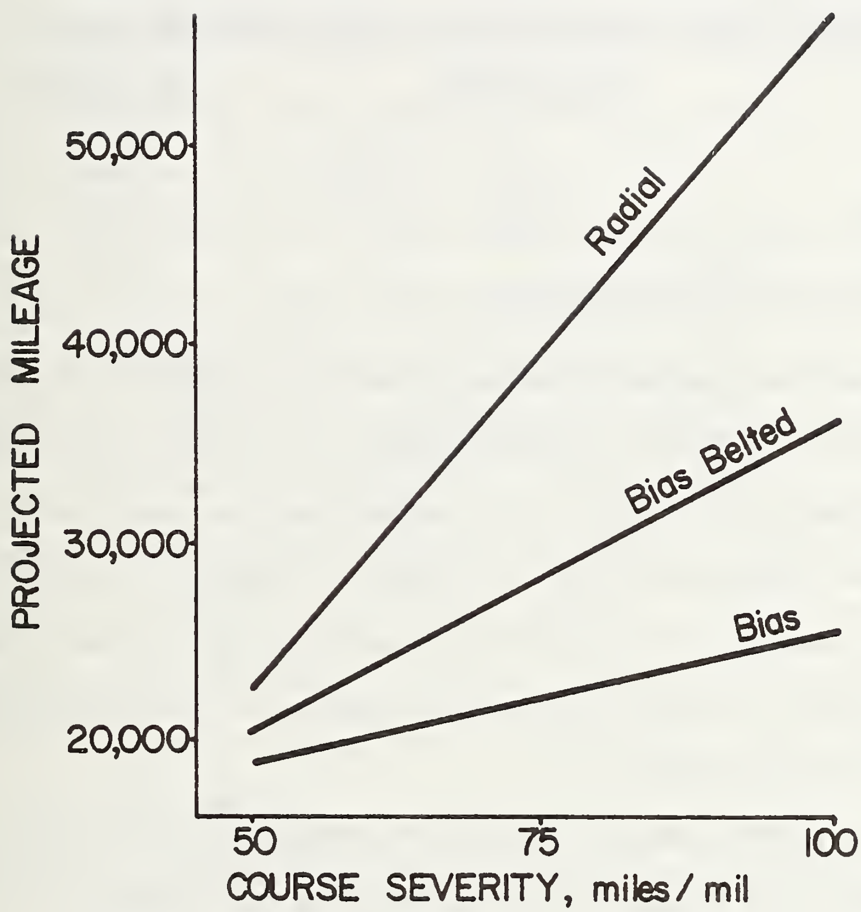
Determine pavement reflection coefficients for typical pavements as a function of incidence angle, particularly for near-grazing incidence, as for near-ground-level sources. This task is probably within the scope of a one man-year effort.

5. Examine the manner in which multiple-tire sources (as in twin dual mountings) interact to produce a composite noise level and noise directionality pattern. Again, this task should be considered a one man-year effort subject to continuation after pay-off demonstration.

4.3 Tire Wear

On conventional surfaces, tire wear appears to be largely a function of tire material, design and construction as may be concluded from Figure 4.7 [Brenner and Kondo, 1973]. However, there is no question that surface characteristics affect the rate at which tires wear. The literature contains, however, little conclusive evidence on this relationship. The reason for this is that tire wear is a slow process, that it is dependent on many variables, but mostly because no experiments have been conducted with the objective of isolating the influence of surface characteristics. When tire wear is studied, this is being done to determine the effects of tire properties, vehicle characteristics, and operating variables on the wear. Even then considerable difficulties are encountered. The most influential variables are the severity of the test course and the performance of the test driver. Speed, speed changes and cornering affect the wear rate. Even if these factors are controlled within tight tolerances, environmental factors introduce considerable variability.

These factors, such as temperature, rainfall and sunshine (which affects pavement temperature) are not easily generalized. Some rubber com-



1 mile = 1.61 km

1 mile/mil = 1.61 km/0.025 mm

Figure 4.7. Idealized plot of effects of course severity on projected mileage by tire construction (increasing abscissa indicates decreasing severity). [Brenner and Kondo 1973]

pounds show increased wear rates with increased atmospheric temperature, while for others the opposite is true [Bulgin and Walters, 1968]. This suggests that the rate of wear may be highly dependent on the interaction between rubber compounds and the several environmental factors. Various abrasion experiments support this view, but the results are not easily transferred to tire wear data analysis. Brenner, et al., 1971 report the following results which indicate that, in addition to being dependent on tire type, tire wear rates are higher during wet than during dry periods:

Average Rates of Wear for Dry and Wet Periods, Mils/1000 Miles(1)

	Dry periods	Wet periods
Tire types		
Bias ply	21.0	25.7
Belted bias	13.4	18.9
Radial	11.0	14.0

(1) 1 mil/1000 mi = 0.025 mm/1600 km

There are several mechanisms by which tires wear [Bergman et al. 1973]. These include wear due to wheel slippage, abrasive or cutting wear, caused by sharp edges and abrasive materials on the pavement surface, wear due to rubber reversion or thermal depolymerization, and cohesive tearing or wear by roll formation. The existence of this mechanism is evidenced by the fact that cigar shaped particles of rubber are formed in samples of loose particulate material collected from pavements.

Slippage wear is most commonly identified with smooth microtextured surfaces when there is prolonged sliding and slipping [De Vinney 1967]. Temperature effects appear to be strong in wear exhibited as smearing and cracking [Holmes, et al., 1972].

It is conceivable that abrasive wear and cutting occur on aggregates having sharp edges and acute-angled asperities (<90°). They can cause high local stress concentrations (on the order of 1000 psi or 6895 kPa) [Sabey 1969], resulting in cuts and ruptures of the rubber and subsequent tearing.

Rubber reversion is depolymerization due to excessive local temperatures. It occurs only when the tire is sliding or slipping and does not rotate. This action leads to blistering or probably to a smooth, shiny surface skin which has low resistance to abrasion.

To investigate the mechanisms of tire wear, various laboratory experiments have been or could be conducted, but on a pavement no one type of wear exists by itself. Tire tests to isolate the types of wear while secondary variables are held constant would be very costly to conduct chiefly because of the difficulty of measuring tire wear in small increments. Therefore, one is forced to make generalizations and simplifications.

Since it is assumed that abrasive wear increases with increasing microtexture and friction increases with increasing microtexture, one can state that the better the friction properties of a surface are, the higher the tire wear will be, Figure 4.8 [Bond et al., 1974, Lees and Williams, 1973].

It is further shown in Figure 4.9, from [Sabey 1969], that microtexture is the most important variable influencing tire wear under dry conditions, and that macrotexture does not, in general, influence the rate of tire wear [Sabey 1969, Lees and Williams 1976]. It is seen from Figures 4.8 and 4.9 that the tire wear increases consistently with microtexture, while no significant effect of macrotexture is evident.

Laboratory techniques for evaluating tread wear under well controlled conditions treat macrotexture surfaces with variable microtexture. The Variable Speed Internal Machine [Lees et al. 1974] has the capability to test coarse macrotexture including open graded asphalt friction courses, but the tire wear tests reported were conducted at two levels of microtexture with similar results as those by Sabey (Figure 4.9). They conclude, in agreement with [Lowne 1969] that macrotexture does not in general influence the rate of tire wear. It must be remembered, however, that they have only considered dry surfaces. Recently

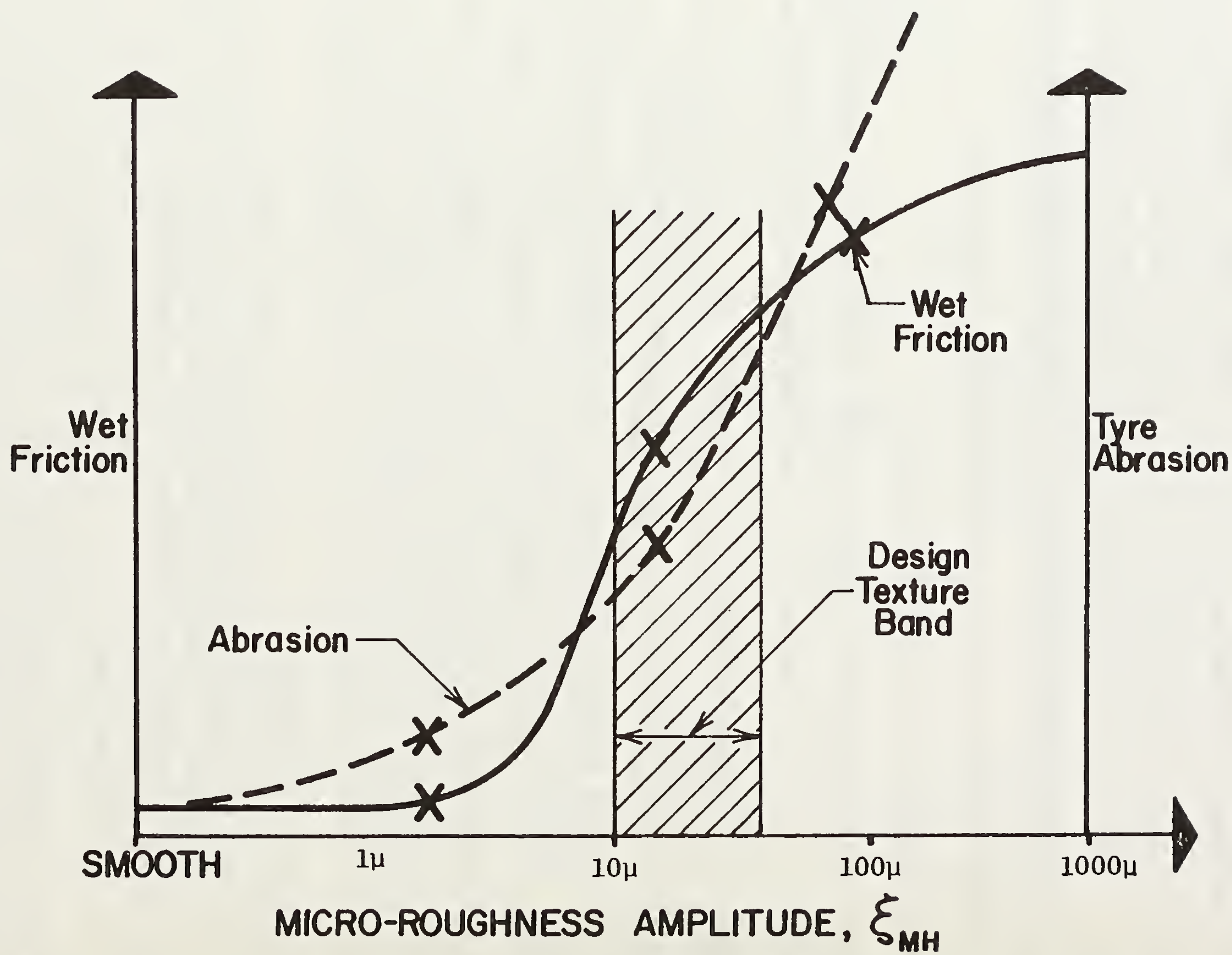


Figure 4.8. Tire Wear [Lees and Williams, 1973 and 1976]

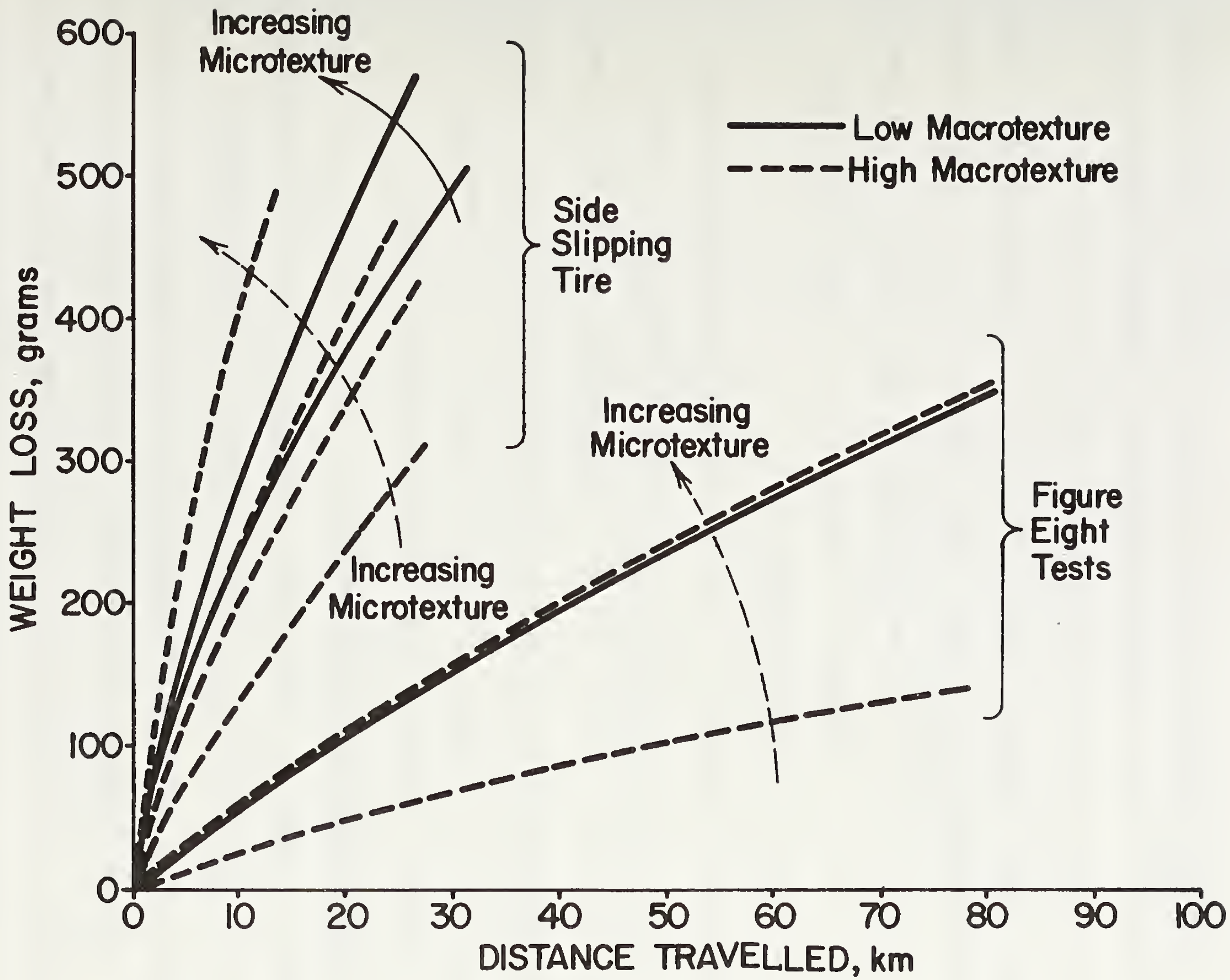


Figure 4.9. Wear of Tyres on Surfaces of Different Texture (Tests Dry) [Sabey 1969]

developed techniques for tire wear testing have been reported [Gusakov et al. 1977], but while the method has been demonstrated, there is insufficient data from which to draw conclusions at this time.

Thus, one can broadly generalize that for pavements with acceptable friction characteristics, tire wear will tend to increase with available skid resistance. However, based on the data available to date, it appears that it is more desirable to achieve high skid resistance by providing better macrotexture rather than by increasing microtexture, since this will result in lower tire wear rates. For example, a macrotexture sand patch depth of .070 inches (1.8 mm) would be desirable to obtain a skid number of 50 at 40 mph (64 kmph). The aggregate that provides adequate microtexture in conjunction with this macrotexture should be used. An example of quantitative, though insufficient, information on tire wear appears in Table 7.7.

Recognizing the importance of tire characteristics relative to their ability to provide traction and resistance to wear, including temperature effects, the U.S. DOT National Highway Traffic Safety Administration (NHTSA) has recently announced new regulations for uniform tire grading that will help consumers to buy the best tire for the money. The announcement in a news release on July 14, 1978 states that the regulations will become effective in 1979.

4.4 Rolling Resistance and Fuel Consumption

A literature search was conducted to determine the effect that aggregates have on tire rolling resistance. From the effects of rolling resistance on total energy vehicle loss, the effect of aggregates on fuel consumption was then determined. The literature shows that the energy consumption of pneumatic tires is comparable to that of the muffler, air cleaner, emission control devices, etc., only 5 to 10 percent of the total car losses. Even though rolling resistance increases with speed, at higher speeds (where car aerodynamics becomes important) it contributes an even lesser percent of the total energy loss, since wind losses increase at a greater rate. Also, it was found that aggregates contribute about 10 percent to the rolling resistance, and when this is multiplied by the 5 to 10 percent of energy loss due to rolling resistance, one finds that the overall energy loss due to aggregates is less than 1 percent. Details of the main references are given below.

4.4.1 Fuel Consumption

Gough [1974] presented an equation of the rolling resistance of a tire, given as the sum of internal and interface forces.

$$D = C(1/\gamma - 1/h) + Wa/h$$

where D is the effective drag force representing the internal losses in the tire plus the tire-pavement interface losses,

C is the input torque to the wheel,

W is the load,

a is the offset of center of pressure from the mid-point of contact,

h is the axle height,

γ is the effective rolling radius (the distance the axle travels per radius of rotation).

Stiehler [1960] deals with the subject experimentally and concludes that the effects of speed, load and inflation pressure on energy loss vary with the design, construction, and type of rubber. Similarly, Clark [1975], in a DOT study in which he gives a good overview of the various effects (load, slip angle, tire type, cord type, speed, etc.) on rolling resistance, concludes that tire construction is the main key to reducing energy losses.

Bowden [1964] states that rolling resistance is caused by deflections of the road surface and the tire due to load or road roughness. However, the deflection of the tire due to aggregates is small (less than 10 percent) as compared to the deflection due to road roughness or to the load of the vehicle; in fact, deflections due to aggregates are more like that of texture than road roughness. Walter [1974] similarly shows that tire rolling resistance is of the order of 10 to 20 lbs per 1000 lbs of load (145 to 290 N per 450 kg) and states that the principal cause is hysteresis in the tire. Walter [1974] attributes this to the fact that for every inch (2.54 cm) the tire sinks into the surface or wraps around the roughness, about 30 lbs (440 N) of resistance for every ton (900 kg) of load must be added to overcome the hysteresis forces in order to move the vehicle. It is not necessary for the tire to penetrate the road surface for rolling resistance to increase, if the road surface distorts appreciably under load. His conclusion on the effects of the road surface on rolling resistance is that rigidity and large-scale roughness are the main factors.

A recent test by DeRaad [1978] showed an average effect of texture on rolling resistance to be of the order of 10 percent, except in one case in which a seal-coated asphalt gave a 33 percent increase. However, this was a single test point, and since no other researcher found a percentage higher than 10 percent, it is difficult to accept this single value.

The conclusion to be reached from these studies is that rolling resistance is mainly affected by tire construction, type of cord, rubber compound, and inflation pressures. While road roughness can be a factor as compared to deflections due to load, the aggregates are on the scale of texture rather than roughness, and contribute less than 10 percent to rolling resistance.

4.4.2 Fuel Consumption

A few papers dealing with total energy loss or fuel economy are included, since fuel consumption and energy losses are directly related. Schuring [1974] presents an equation of the power required for a cruising auto:

$$P = 1/375\ (F_x + F_r)(R_{1r} + k)[1 + (F_x + F_r)C_s]\ V/R_{er}$$

where P is the power required by driver tire,

F_x is tractive force,

F_r is rolling resistance force,

R_{lr} is loaded tire radius,

k is rolling resistance moment constant,

C_s is braking stiffness,

V is automobile velocity,

R_{er} is effective rolling radius.

Values from this equation show that the energy consumption of pneumatic tires is comparable to that of the muffler, air cleaner, emission control devices, etc., (all in the 5 to 10 percent loss range). Similarly, Crum [1974] concludes that the energy losses due to tire rolling resistance seldom exceed 10 percent of the fuel consumption of passenger cars.

4.4.3 Combined Effect

In general, there is agreement in the literature that tire rolling resistance is caused by (1) hysteresis, the largest contributor, responsible for up to 95 percent; (2) friction (or scrub between tire and road) causing 5 to 10 percent of loss; and (3) tire windage drag accounting for only 1.5 to 3 percent. Aggregates do play a role in the hysteresis losses; however, their role accounts for only about 10 percent of the hysteresis losses. Aggregates also affect frictional losses; however, when the effect is added to the hysteresis losses, the total effect of aggregate on rolling resistance is less than 10 percent, or less than 1 percent of total losses. Thus, for this study, their role in fuel consumption will be ignored.

4.5 Light Reflection and Glare

Generally, daytime driving poses no serious problems related to light reflection except when a wet pavement reflects dazzling sun rays. The problems of light reflection and glare are, therefore, principally related to night visibility, particularly on wet pavement surfaces. Early work on light reflection [Christie 1954] was mainly concerned with the street lighting case where high luminance (brightness) produced on illuminated streets and intersections is desirable. In this context, coarse-textured surfaces used to improve skid resistance were found not desirable because they cut down the reflection of obliquely incident street lighting, thereby reducing luminance.

As wet weather problems had to be solved by providing compatible requirements, concern with the problems of roadway lighting by traveling vehicles became more prominant. Perhaps the most informative report on this aspect of the problem is in a paper by Fredsted [1965]. In the paper, Fredsted discussed the problem of light reflection on Denmark's pavements at night in some detail. He stated that light reflections may either be diffuse preferential or retro-reflective, and that the latter was not applicable to road surfacings. Fredsted stated further that preferential reflections may cause dazzling and uneven distribution of luminance which can be avoided only by using a coarse textured surface that will permit surface asperities to penetrate the water film which causes mirror-like effect. Fortunately, this solution is in accord with providing coarse-textured surfaces for improving skid resistance. Figure 4.10 [Fredsted 1965] indicates that the least preferentially reflecting surfaces offer the highest skid resistance. Fredsted [1965] also found that diffuse reflective qualities of light decrease substantially when the road surface becomes wet.

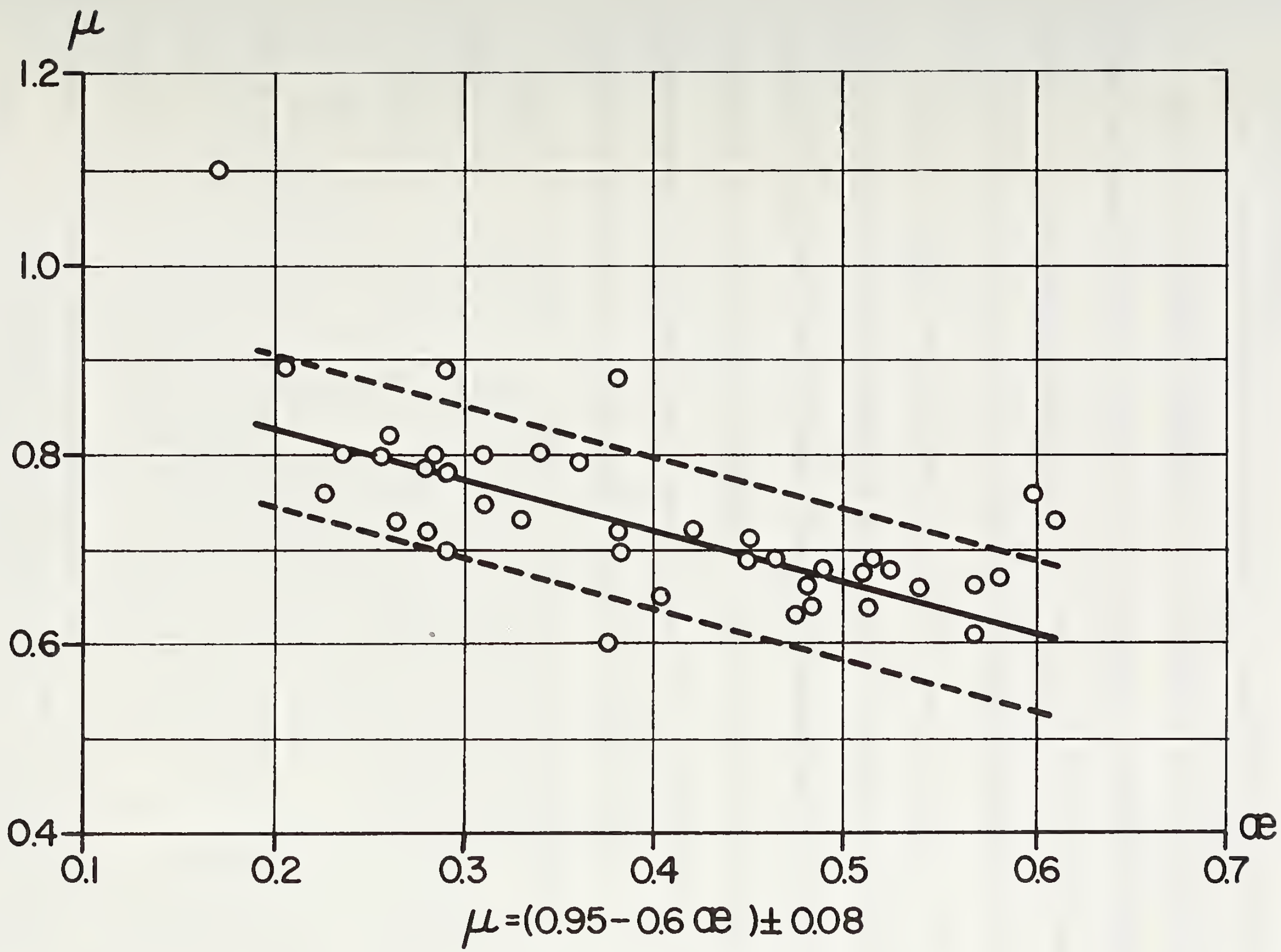


Figure 4.10. Correlation between Skid-Resistance μ and Reflective Quality æ [Fredsted 1965]

To alleviate this problem, the use of light-colored aggregates in bituminous surfaces was found effective. Fredsted [1965] reported that a blend containing 30 percent light-color coarse aggregate (e.g., synopal) in a darker mass was found sufficiently effective in producing color contrast to assist the driver in perceiving the roadway to a safe distance (50 to 100 meters), particularly if the light-color aggregate protrudes 0.6 mm or more above the darker aggregate. Furthermore, it was found (Fredsted 1965] that this color-contrast effect applies to white stripes as well.

Following a study of dry road surface reflection characteristics in Britain, Sabey [1972, page 8] concluded that "there is an urgent need to investigate further the problem of reflection characteristics of wet roads, with a view to formulating recommendations for surface texture requirements to improve the uniformity of luminance." However, in a subsequent publication [Sabey 1973, page 10), the same author concluded that "for improvements in skidding resistance and visibility by day and night, all evidence points towards the need for macroscopically rougher textures. Additional particular requirements are: for skid resistance--harshness; for night visibility--harshness and angularity of projections; for general visibility--porosity of surface." Marek et al. [1972, page 48] stated that "Even though most conventional aggregates have little reflectivity, reflective aggregates are desirable to contrast the road surface and its surroundings" provided that "aggregate particles on the surface of a pavement should not create a glare condition."

To sum up, it may be concluded that wet-road visibility and glare reduction may be achieved by incorporating light-colored, non-shiny (non-glassy) aggregates in a coarse-textured, porous pavement surface.

A FHWA project entitled "Determining Pavement Reflectivity for Roadway Lighting Design" is planned to start in 1978 under a request for proposal, RFP 215-8. An objective of the proposed research is to relate light reflection and pavement texture.

4.6 Splash and Spray

Drivers will agree that splash and spray constitute an important problem which impairs visibility, especially at night. Accordingly, it should be obvious that any improvement which contributes to the lessening of splash and spray is a positive contribution to driving safety.

Maycock [1966] defined splash as "the large droplets of water which are thrown off the tyre or squeezed out from the area of contact between tyre and road" and added that splash has been associated with large water depths or low speeds. He defined spray as the "very fine envelope of spray which is carried in the turbulent air stream around the vehicle," and added that spray is associated with shallow water depths or high speed.

Several investigators in the United States, the United Kingdom, and other countries have studied various types and configurations of mudflaps to alleviate the problem of vehicle splash and spray on wet surfaces, but only few attempted to vary the surface texture. In fact, the only significant published work on this aspect of the problem that has been found by the authors in the available literature was done by two British investigators [Maycock 1966 and Brown 1973]. Performing tests on a "standard fine, cold asphalt" surface, Maycock found that spray was negligible below 30 mph (48 kmph), but increased substantially with increasing speed. In the range of 45 to 75 mph (72 to 120 kmph), spray density varied with speed according to the following equation:

$$\text{Spray density} = \text{Constant X (speed)}^{2.8}$$

He also made tests on six bituminous surfaces of which four were impervious, one slightly pervious, and one very pervious (porous). Maycock [1966, page 18] concluded that "excluding the very porous macadam carpet, the spray generated on the other fine surfaces did not vary by more than a factor of 2. The surface dressings were slightly better than the smoother asphaltic surfaces. The very porous macadam carpet was outstandingly good in reducing spray as virtually all the water falling onto the surface very quickly drained into it." Results of the experiments made on the six surfaces at 60 mph (96 kmph) are shown in Table 4.4 [Maycock 1966, page 8].

Brown [1973] described six experimental open-textured bituminous-macadam pervious surfacings of which four had 19 mm and two had 10 mm nominal top size coarse aggregate. All the surfaces were constructed using the same coarse and fine gritstone aggregate and 2 percent hydrated lime as part of the filler. Table 4.5 [Brown 1973, page 5] shows aggregate gradation and bituminous (asphalt) content for the test sections. Other construction information and test data are detailed in Brown's report. His principal conclusion is that all six experimental pervious surfacings have retained their spray-reducing properties after being subject to heavy traffic for almost two years, even though there was 20 to 50 percent reduction in permeability and about 33 percent reduction in air voids. Brown points out that the sections had proper cross-slope to prevent water puddles (which cause splash) and that air-voids after traffic compaction were on the order of 20 percent.

The cited work of Maycock and Brown and observations by others [AASHTO Guidelines 1976 and Cram 1975] point out the significance of open-graded bituminous pavement surface texture and well-textured PCC in reducing splash and spray as well as reducing glare, skidding risk, and hydroplaning potential of wet surfaces at high speeds.

Table 4.4. Results of the Experiments on the Effect of Surface on the Amount of Spray Produced, Including Details of the Surfaces [Maycock 1966]

	Surface	Details [1]	Average Quantities of Spray Collected (g)	
			Expt 1	Expt 2
1	Fine cold asphalt	Cold asphalt to B.S. 1690 using blastfurnace slag aggregate - impervious	87	93
2	Asphalt with chippings	Hot rolled asphalt to B.S. 594 with 1/2-in. meldon white chippings rolled into surface (120-140 yd^2/ton) - impervious	111	133
3	Bridport surface dressing	A surface dressing made with 3/8-in. rounded gravel - impervious	76	71
4	Meldon surface dressing	A surface dressing made with 3/8-in. meldon white chippings (100-120 yd^2/ton) - impervious	66	75
5	Mixed aggregate	Bitumen-macadam to B.S. 1621 with 3/8-in. aggregate of 50% rounded gravel and 50% crushed quartzite rock - slightly pervious	-	107
6	Quartzite	Bitumen-macadam to B.S. 1621 with an aggregate of 3/8-in. crushed quartzite rock - pervious	0	-

1. 1 in. = 25.4 mm
yd^2/ton:1 yd^2 = 0.84 m^2; 1 ton = 907 kg

Table 4.5. Specified Composition of Macadams [Brown 1973]

Aggregate Grading Percent by Weight Passing [1]	19 mm Nominal-Size Bitumen Macadam	10 mm Nominal-Size Bitumen Macadam
1 in. BS sieve	100	
3/4 in. BS sieve	90-100	
1/2 in. BS sieve	50-80	
3/8 in. BS sieve		90-100
1/4 in. BS sieve	25-35	40-55
1/8 in. BS sieve	10-20	22-28
No. 200 BS sieve	3-6	3-5
Binder Content	4.0-4.4	4.4-4.8

1. 1 in. = 25.4 mm

4.7 Other Characteristics

In bituminous mixtures, it is important that the aggregate will be able to stand plant mixing temperatures of 300-400 F (150-200 C) without being subject to decomposition, weakening or any other deleterious changes. In any mixture, the surface aggregates must be stable and durable for normal surface life environmental conditions and maintenance practices, such as freezing and thawing, wetting and drying, salt deicing, and other possible effects due to weathering [Marek, et al., 1972].

Not as critical but important are characteristics such as the ability of the aggregate to conduct heat and electricity so that the pavement surface will neither become so hot as to melt rubber tires nor allow the build-up of high static electricity charge caused by rubbing between rubber and pavement.

It is beyond the scope of this report to go into more detail about these and other extraneous but related surface characteristics. However, some details are available in literature concerned with this type of surface aggregate characteristics [Marek, 1972 and others].

4.8 Preliminary Conclusions

Based on the preceding discussions of the various surface parameters and factors involved in constructing the surfaces, one may conclude that a surfacing system is considered optimal only if it will result in an overall satisfactory and safe performance at the least cost, depending on the intended use and conditions for the system. If the surface either falls short of satisfying usage requirements or is uneconomically overdesigned for the conditions of its use over the expected surface life, then it should not be considered optimal. Salt [1976] proposed four pavement site categories based on skidding accident risk levels. The four categories were termed: very difficult, difficult, average, and easy. Different traffic conditions must also be combined with each of the site categories when considering an optimum design for a given site. The site categories were defined as shown in Table 4.6.

In the "easy" performance category, no problems will be anticipated whatever the type of pavement or aggregates used, as long as they meet commonly used conventional specifications and construction practices. For example, if the most economical aggregate available is limestone or dolomite, it should be satisfactory for light traffic when it is the only aggregate used, and for medium and heavy traffic when either blended with higher quality aggregate or when the fine aggregate is of a better quality. On the other hand, using excess asphalt in the mix that may cause bleeding is not satisfactory for any use. In the "average" category, most conventional surfaces will be satisfactory if conventional specifications and construction practices are carefully observed. However, in this category the aggregate must be of at least medium grade, such as granite, gneiss or other material of a history of a good performance, as often is the case in carbonate aggregates with high sand-size silicious content and in some slates and shales of proven satisfactory performance. In the "difficult" category, the best available conventional designs should be utilized in conjunction with careful observation of construction practices. These include high strength PCC with a high proportion of sand within the specification limits, "such that an adequate amount of mortar for texturing is produced on the surface without the addition of water or excessive manipulation" according to AASHTO [1976].

Table 4.6. Site Definitions
Adapted from [Salt 1976]

Site	Definition	Suggested $sfc_{50\ km/hr}$ 1/
1. Very Difficult	(i) Approaches to traffic signals on roads with a speed limit greater than 40 mile/h (64 km/h) (ii) Approach to traffic signals, pedestrian crossings and similar hazards on main urban roads	0.55 - 0.75
2. Difficult	(i) Approaches to major junctions on roads carrying more than 250 commercial vehicles per lane per day (ii) Roundabouts and their approaches (iii) Bends with radius less than 150 m on roads with a speed limit greater than 40 mile/h (64 km/h) (iv) Gradients of 5% or steeper, longer than 100 m	0.45 - 0.65
3. Average	Generally straight sections of and large radius curves on: (i) Motorways (ii) Trunk and principal roads (iii) Other roads carrying more than 250 commercial vehicles per lane per day	0.30 - 0.55
4. Easy	(i) Generally straight sections of lightly trafficked roads (ii) Other roads where wet accidents are likely to be a problem	0.30 - 0.45

1/ sfc - Sideway force coefficient (of friction)

Polish-resistant coarse aggregates should also be used, at least in the top 1-2 in. (25-50 mm) near the surface. The mortar surface should be well-textured, preferably transversely, using metal tines or steel or fiber brooms [AASHTO 1976]. "Best" conventional designs also include high quality plant mix, hot laid dense-graded or open-graded asphalt concrete. High-quality AC designs are generally well-known to highway departments and paving agencies, but improvements are being made through experience and experimentation with modifications. Typical gradations for dense and open-graded mixes have already been discussed (Table 3.1). In this category of "difficult" sections, only high quality natural or synthetic aggregate should be used in the surface course (top 1-2 in. or 25-50 mm). For example, durable crushed sandstone, arkose, graywacke, and high friction quartzite, or selected durable and crush-resistant lightweight aggregate such as some expanded shales, clays or slags. Where these materials may not be economically available, high-type igneous and metamorphic aggregates such as granite and gneiss may be used if blended with better performing aggregates. In this category, it is also recommended that natural siliceous sand or a similar well-performing material be used for the fine aggregate portion. Finally, in the "very difficult" category, only the "best" design and highest quality aggregates and well-controlled construction practices will result in optimum pavement surfaces. Aggregates for these locations must be of the highest performing naturally occurring or synthetic types such as proven high-quality, wear-resistant, and skid-resistant sandstone, graywacke, and quartzite, or when possible, emery or synthetic aggregate that may have been proven of superior quality, such as calcined bauxite, calcined flint, or some ceramic materials. Based on results of work with hard granules reported by Hosking [1970], one alternative may be blending surface mortar sand in PCC (the top 1/4 to 1/2 in. or 6 to 13 mm) with 20 percent aluminum oxide or silicon carbide granules of a medium size range gradation (#16 to #100 sieve) or sprinkling the granules and pressing them into the surface before texturing. However, more experience is needed to prove how effective this procedure may be. It is also recommended that high strength binders be used in this category to hold the high quality aggregate particles in place for reasonably long periods under continuous, high stress conditions. It is in the "very difficult" category that providing satisfactory long lasting surfaces needs more research with the objective of finding more innovative aggregates and surfacing systems at economical costs. In exploring possibilities of producing high-performing, wear-resistant and polish-resistant aggregate, target values of properties that should enhance the process are included in Table 3.7. These values are based on high but attainable goals reported in the literature.

It must be remembered that, in any category, performance must meet all the important requirements of the given situation. For example, where noise abatement is of high priority, friction requirements associated with noisy textures or coarse mortar texturing and transverse grooving may have to be compromised, requiring possible compromises in other aspects such as lowering the traffic speed. Lowering speed may also be required for reducing tire wear and fuel consumption, until new and innovative aggregates and surfacing systems are found that provide high friction without having to utilize harsh surface textures. At the present level of knowledge, it does not appear that a quantitative satisfactory model or models can be established to satisfy simultaneously all performance requirements that demand conflicting aggregate and surfacing system characteristics. For this reason, optimized trade-offs and compromises must be utilized.

5.0 PROCESSES TO MODIFY AND SYNTHESIZE AGGREGATES FOR SURFACE MIXTURES

Synthetic aggregates are produced by the thermal or chemical processing of raw materials that may be either natural or man-made. Synthetic aggregates include waste materials as well as those materials synthesized especially for use as skid-resistant aggregates. Potential synthetic aggregates vary considerably in their physical characteristics depending on the raw material and the processing method used in their manufacture. In many cases, their physical characteristics and engineering behavior are considerably different from natural aggregates upon which current specifications and test methods are based. Because of the wide variety of synthetic aggregates that can be produced, it is convenient to classify modified or synthesized aggregates into groups. This may be done in a variety of ways, according to method of processing, physical properties, raw materials, pavement application, etc. [Roy 1977, Fondriest and Snyder 1964, Marek et al. 1972].

5.1 Review of Ceramic Processing

The field of ceramic processing is quite varied and has developed into a highly specialized industry. Examples of ceramic processing are found in brick manufacture (common and refractory), pottery making, grinding wheels, pyroceram, and ceramic coatings, [Kingery 1960]. A wide variety of traditional processing techniques have been developed to meet specialized product requirements. A list of these techniques is given in Table 5.1 where four different ceramic processing steps are identified:

1. Material preparation, I
2. Mixing, II
3. Forming, III
4. Consolidation, IV

Ceramic Type A (see Table 5.1) refers to traditional methods of ceramic processing such as the manufacture of clay pottery. The consolidation step involves sintering or bonding, in which the fine-grained raw material reacts in the solid state to form a monolithic mass. Ceramic Type B, Liquid Phase Sintered, differs from A in that liquid rather than solid phase sintering is employed. In both instances, some residual porosity remains after firing. The amount of porosity is related to the firing temperature; lower temperatures yield higher porosities. The porosity is due to incomplete consolidation of the raw material.

Types C and D both involve the melting of the raw material with subsequent casting into a mold. Some additional heat treatment is usually specified (annealing) to control residual thermal stresses in the finished product. Both C and D differ from A and B in that complete melting is required, in A and B the raw material is heated to a temperature below its melting point.

Although the glass-forming process is, in itself, of little interest in aggregate production, D.1 crystallized glasses are relevant. Pyroceram and blast-furnace slags are good examples of crystallized glass. The raw material is first formed by complete melting and is then solidified in the amorphous state. By reheating or holding for prolonged periods at elevated temperatures, crystal growth, referred to as ceraming, can be promoted, producing

Table 5.1. Ceramic Processing

Ceramic Type	Processing Steps: I Raw Material Preparation	II Mixing	IIIa Forming	IIIb Forming	IV Consolidation
A: Normal	Yes	Yes	Slip Cast Plastic Cast Jiggering	Drying	Sintering
B: Liquid Sintering	Yes	Additive to give liquid phase	As above		Liquid Sin-tering
C: Fusion Casting	Yes	Yes	Cast in Mold		Anneal
D.1 Glass Forming	Yes	Yes	Pour into Mold		Melting
D.2 Crystallized Glasses	Yes	Yes			
E: Pelletizing	Yes	Yes	Binders Extrude Rotary Kiln	Drying	Sinter Calcine
F: Coatings	Yes	Yes	Hot spray cold solu-tion Plasma spray		None

ducing a glass ceramic, i.e., a ceramic with a glassy matrix. This technique has been highly refined in the production of pyroceram; it occurs by happenstance in the air cooling of blast-furnace slag. The ceraming step produces a dense, intertwined network of crystals which yields excellent strengths. The extent of the residual glass matrix is controlled by the composition of the melt and the temperature schedule during the ceraming process. The glass phase may predominate or it may be as small as a few percent.

Type E is also of importance in aggregate manufacture and is the process whereby lightweight sintered clays and shale aggregate is produced. The agglomerated material is passed over sintering grates or passed through a rotary kiln where it is heated sufficiently to sinter the intimately mixed mineral grains into a monolithic mass. The agglomeration may occur in nature as with shale, it may occur in the kiln as with clays, or it may be done in a separate pelletizing or extrusion step as with aggregate made from sintered fly ash. Often a binding agent is added to give adequate "green" strength and sometimes a bloating agent is added as with some expanded clay lightweight aggregate.

Type F involves the addition of coatings to a substrate. These techniques have been developed for specialized industrial purposes and are technology and cost intensive. Chemical Vapor Deposition (CVD) and flame spraying are possible methods of applying coatings to aggregate materials. The cost of application is in the range of $10-100/pound ($5-50/kg) [Roy 1977], which makes their viability as methods for aggregate production or beneficiation exceedingly suspect. Other approaches would be the use of a low-temperature coating or the use of a coating that can be applied before firing, much as glazes are produced on ceramic ware (e.g., pottery). Still another approach [Roy 1977] is the use of a low-melting, fluidized spray that contains a premixed fuel.

Many of the processes identified above are energy intensive due to refinements in the processing technique as for example, the production of crystallized glass (pyroceram). The extended thermal treatment needed to produce crystallization is more energy demanding than the sintered process where the raw material is brought to a temperature below melting. Also to be considered is the energy content of the raw feedstock. For example, coal mine refuse, fly ash and other refuse materials contain some unburnt carbon and this can be used to feed the ceramic processing. Likewise, if the feedstock is a molten slag, the cost of initial fueling can be eliminated. This could be realized if certain industrial slags, such as boiler slag, coal gasification wastes, and pyrolysis slags, were tied directly to the production of aggregate, rather than disposed of in the most convenient manner and later reclaimed as aggregate. The resulting energy savings can be considerable. For example, the energy requirement to sinter fly ash is about 1,000,000 BTU/ton (1200 kJ/kg) whereas energy requirement for sintered clay may be of the order of 3-4 million BTU/ton (3500-4600 kJ/kg), because of the carbon content of the fly ash [Faber 1976].

The energy requirements for a number of ceramic processes are given in Table 5.2 [Whittemore 1974]. The energy requirements and kiln efficiencies vary considerably according to the load on the kiln, the quantity of heat, and the temperature required. The rotary kiln as used in the calcining of dolomite is quite efficient, 63 percent. The tunnel kiln, in which the green ware passes through the kiln in a wheeled car, varies considerably in efficiency depending on the load in the kiln (compare 15, 16, 17 with 1, 2). Fusion processes, while surprisingly efficient, command higher temperatures and have larger energy requirements (21-25 under Abrasives and Special Refractories). The relative energy saving from coal-mine refuse and fly ash can also be seen (26-28).

Table 5.2. Selected Values of Energy Required and Used in Various Ceramic Processes [Whittemore 1974]

Process	Temperature of process (°F)(1)	Energy, Btu/lb(2)	Efficiency (%)
Structural Products			
1. Top fired tunnel kiln (includes drying)	2100	1200	68
2. Side fired tunnel kiln (includes drying)	2100	1700	48
3. 32 ft. periodic, IFB lining(3)	2000	2100	30
4. 32 ft. periodic, firebrick lining(3)	2000	4300	15
5. Kettle calcining gypsum to plaster	305	550	51
6. Drying plaster board	210	1132	56
Refractories			
7. Periodic kiln firing fireclay retorts	1900	8500	7
8. Tunnel kiln - fireclay brick	2600	2250	35
9. Rotary kiln calcining kaolin	3000	3000	32
10. Rotary kiln calcining dolomite	2000	3200	63
11. Rotary kiln dead burning dolomite	3180	4400	53
12. Rotary kiln sintering magnesia grog	3270	6750	29
Glass Containers			
13. Gas fired regenerator furnace	2600	2820	39
14. Electric furnace	2600	1700	64
Whitewares			
15. Periodic kiln - porcelain	2650	43000	2
16. Tunnel kiln - porcelain	2570	18000	5
17. Tunnel kiln - tableware	2240	3000	27
18. Decorating kiln - tableware	1350	1000	45
19. Direct fired tunnel kiln - sanitary ware	2160	5000	15
20. Recuperator tunnel kiln -sanitary ware	2160	3000	25
Abrasives and Special Refractories			
21. Arc fusion of magnesia	5200	4800	50
22. Arc fusion of zirconia from zircon	5000	10000	49
23. Arc fusion of alumina	3800	2400	64
24. Reduction to silicon carbide	3600	12500	71
25. Reduction to boron carbide	4350	50000	52
Synthetic Aggregates (Estimated Values)			
26. Sintered shale, clay	2200	2500	--
27. Sintered coal mine refuse	--	600	--
28. Sintered fly ash	--	500	--
29. Fusion of municipal waste	4000	2500	--

(1) $1\ F = \frac{9}{5} C + 32$

(2) 1 BTU/lb = 2.33 kJ/kg

(3) 1 ft = 0.3048 meter

A wide variety of industrial ceramic materials are currently being produced for non-paving applications. In considering these materials for use as paving aggregates, one point should be emphasized. These industrial ceramics are often produced for a specific purpose with the appropriate material specification criteria. For use in paving applications, these criteria may have to be reevaluated; paving aggregates, for example, do not have to be refractory. This means processing requirements as well as raw material requirements may be less restrictive for paving aggregates than for some industrial applications. A considerable cost saving may result or a new supply of raw materials may be opened. An example of the latter is calcined bauxite. Low-grade bauxite,which cannot be used as a refractory brick, makes an excellent paving aggregate, superior to an aggregate made with a purer grade of bauxite. Therefore, by using low-grade bauxite, a much wider source of raw materials becomes available at a lower cost [Tubey and Hosking 1972]. Conversely, requirements that are not significant for refractory brick, for example, may become critical for pavement aggregates. Freeze-thaw and impact resistance are good examples.

5.2 State-of-the-Art Review--Aggregate Processing Methods

The processing of high-technology, ceramic synthetic aggregate has been discussed in detail by Roy [1977]. The various methods involve either the agglomeration of fines (heating or thermochemical means) or physical alteration by heating (calcining). Potential sources and processes of synthetic aggregate have also been grouped by Fondriest [1964] as follows: (1) agglomeration of fines by heat treatments, (2) waste materials processed by heat treatment, (3) composite materials including coatings, and (4) chemical or thermochemical processing of fines.

With the exception of some forms of thermal chemical processing, these techniques are energy intensive and will become more expensive as energy becomes more costly. Exact energy requirements are dependent on the chemical composition of the raw material, the processing method and the processing steps [Roy 1977]. For example, Synopal [Dryover 1962] and slagceram [Davies et al. 1970], which are first melted, cooled to a glass and then reheated on a "ceraming" schedule, require more heat than a raw material such as expanded shale, which is sintered in a single step in a rotary kiln. In general, higher quality material is more energy intensive. A flow chart listing the various steps in processing different synthetic aggregate is given in Figure 5.1.

Because raw material costs may represent a considerable fraction of the cost of an aggregate, the use of waste materials as raw material is attractive. This is particularly true if the waste can be processed directly from the molten state without the necessity of remelting the material, thereby effecting a considerable saving in energy and/or processing costs.

Waste blast-furnace slag is perhaps the single most important skid-resistant synthetic aggregate currently in use. Its widespread use may be somewhat fortuitous in that factors that optimize iron production also optimize the slag as a paving aggregate [Josephson 1970]. Also, the annual production of blast-furnace slag is large enough to have generated research to promote its use; thus its advantages as well as its disadvantages are well documented.

Lightweight expanded shale is the second most important synthetic aggregate primarily used for structural concrete and lightweight block manufacture. It is doubtful if its use in pavements would be as fully developed as it is at present if it were not for the primary uses. Except where its use is widespread as a paving aggregate, it is unlikely that the

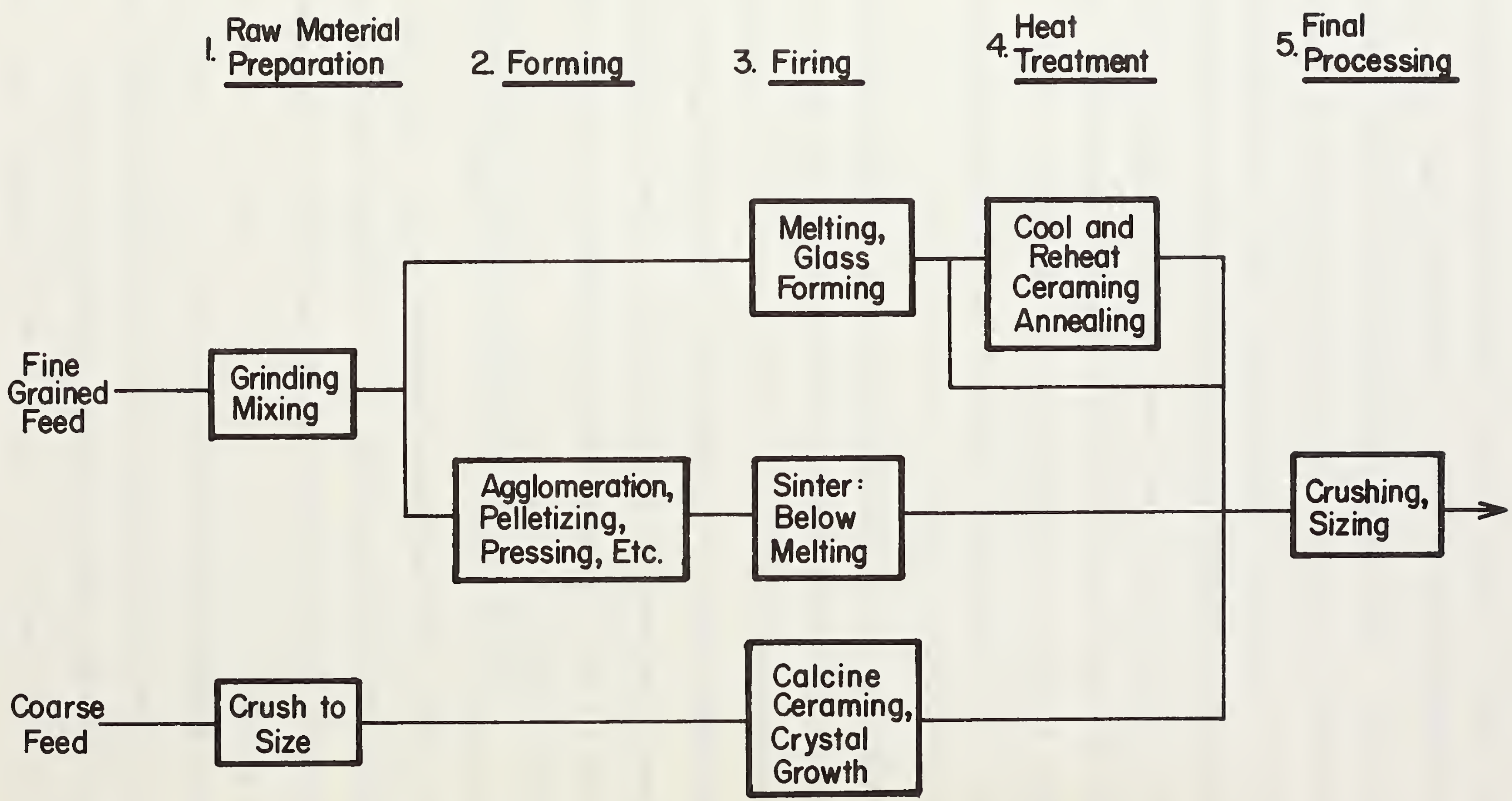


Figure 5.1 A Flow Chart Listing Various Steps in Processing Synthetic Aggregates

development of a new plant could be justified for pavement surfacing applications alone given the volume of aggregate used in surfacing applications.

Marek et al. [1972] noted that there had been few significant additions to the list of available or potentially available synthetic aggregates since 1964. This is still the case in 1977, and increased energy costs, environmental restrictions, and the shortage of money for capital expenditures, have placed additional constraints on synthetic aggregate development. The NCHRP report by Marek et al. [1972] contains a good summary of potential sources of materials for synthetic or supplemental aggregates and is reproduced in Table 5.3. Environmental considerations now limit the utilization of marine deposits, and glass and glass cullet are not satisfactory for use as skid-resistant aggregates. Many industrial wastes, such as the glassy slags from power plants, incinerators, etc., are acceptable for general aggregate use but not as skid-resistant aggregates unless they receive additional processing.

A listing of synthetic aggregates grouped according to method of manufacture is given in Table 5.4. The different methods that appear worth considering relative to aggregate manufacture are:

- 1a. sintering without bloating
- 1b. sintering with bloating (expanding)
- 2a. glass-ceramic without bloating
- 2b. glass-ceramic with bloating
- 3. calcining
- 4. chemical processing
- 5. thermo-chemical processing
- 6. glass making
- 7. glass making with bloating
- 8. coatings

Table 5.3. Methods of Producing Synthetic Aggregates
(After Marek et al., NCHRP Report 135)

1. Heat treatment of suitable clays and shales.
2. Pelletizing and sintering of raw materials.
3. Pelletizing of raw materials and coating.
4. Mechanical processing of certain wastes:
   - (a) Building and highway rubble.
   - (b) Ceramic waste:
     - (1) Devitrified waste.
     - (2) Glass cullet.
     - (3) Waste ceramic ware.
   - (c) Waste from cast-iron enameling.
   - (d) Wastes from structural clay products:
     - (1) Broken bricks.
     - (2) Broken pipe.
     - (3) Broken file.
   - (e) Metallurgical slags.
   - (f) Ash clinker:
     - (1) Incineration of sewage sludge.
     - (2) Incineration of garbage.
   - (g) Scrap iron and steel.
5. Heat treatment of nonbloating materials, including sand, soil, and loess.
6. Manufacture by nuclear methods.
7. Heat treatment of waste materials:
   - (a) Coal mine tailings.
   - (b) Fly ash.
   - (c) Collected mineral dusts.
8. Chemical and thermochemical processing of raw materials or solid wastes from conventional aggregate production:
   - (a) Mixing with agent, pelletizing, and curing:
     - (1) Lime.
     - (2) Lime-fly ash.
     - (3) Cements (phosphate, silicate, portland, etc.)
     - (4) Plastics.
   - (b) Mixing with agent, casting brick, curing and crushing to size.
9. Mechanical processing of marine deposits:
   - (a) Reef shell.
   - (b) Beach sand.
10. Beneficiation of a low-quality aggregate.
11. Combinations of two or more of the previously listed sources.

These methods can be reduced to the following broad categories:

1. Heat treatment of raw minerals.
2. Chemical and thermochemical processing of raw minerals or solid wastes from conventional aggregate production.
3. Pelletizing of raw minerals and beneficiation to suitable quality.
4. Utilization of solid wastes.
5. Beneficiation of low-quality aggregate.
6. Utilization of marine deposits.

Table 5.4. Synthetic Aggregates Grouped by Method of Manufacture

Process	Description	Examples	Current Pavement Use	Comments
1. Sintering	Heat to below melting point agglomerate fines into larger, tougher particles.	Brick making. Sintered shales, clays. "Molocite," Refractories. Fly ash lightweight aggregate.	Texas: "Red Rock"	Agglomerate fine material, e.g., wastes, clays into hard, large-sized particles. Blend ingredients to control differential hardness. Properties vary.
1.a. Sintering w/Bloating	As above except bloating gives "expanded" particle.	Expanded shale, clay, coal mine refuse.	Expanded shale, clay, coal mine refuse.	Good skid resistance, doubtful durability, wear resistance. Control of wear, skid resistance by bubble size density and wall thickness.
2. Glass Ceramics	Controlled thermal processing of glass to give appreciable crystal growth.	Processed blast furnace slag. "Synopal"	Extensive: blast furnace slag, excellent skid resistance	Development of crystalline phase controlled by processing and composition. Desired process waste slags.
2.a. As in 2, but with bloating or expansion	Melted glass expanded with internal gas or by injected air or steam. Appreciable crystal growth on cooling.	Expanded blast furnace slag.	May not be advantageous costwise if process 2 gives adequate differential wear.	Expanded blast furnace slag currently used in block manufacture. No advantage reported over normal blast furnace slag. Cost and environmental aspects disadvantages.
3. Calcining	Heating in solid phase to effect crystal change, drive off H_2O or carbon dioxide	Calcined bauxite	Limited due to expense	Potential as "high class" material. Cost effectiveness needs to be improved.
4. Chemical Processing	Hydraulic cement or other chemical reaction at ambient temperature.	Pozzopac, sulfate-fly ash waste	None	Questionable. Low energy input implies material without hardness and wear resistance
5. Thermo-Chemical Processing	Hydraulic cement or other chemical reaction at elevated temperature.	No known examples	None	Inclusion of hard particles in softer matrix suggested. Process represents low energy agglomeration of fines.
6. Glass Making	Heat to above melting temperature, cool rapidly without crystal growth.	Water quenched boiler slag, steel and blast furnace slags, slags from incinerators, etc.	Boiler slag. Controversial as to skid resistance.	Marginal as skid resistant aggregates. Modify to process 2. if possible. Vesicularity may extend usefulness.
6.a. Glass Making with Bloating	Melted slag expanded with gasses produced in-process or with injected air or steam. No appreciable crystal growth on cooling.	None	None	Vesicularity may improve skid resistance. Probably not preferred process but may be necessary with some waste slag compositions that do not crystallize readily.
7. Recycling	Recycling of PCC or asphaltic pavements.	None	None. Ongoing demonstration projects	Alteration of future construction to allow aggregate recycling should be considered. Crushed PCC may give good differential wear.
8. Coatings	Thermally or chemically applied thin surface coatings.	No known examples.	None	Limited application unless coating thick as in composite material. Expensive processing.

Also to be considered as synthetic aggregates but not strictly associated with ceramic manufacture are:

9. recycling of old pavement materials
10. use of waste materials.

A detailed consideration of each of the above methods of manufacture is given in the following sections. With the single exception of lightweight aggregates from shale and clay there are almost no examples of manufactured synthetic aggregates that are being used as skid resistant aggregates in significant quantities. Blast-furnace slag is an industrial by-product used widely as an aggregate,but neither it nor ordinary lightweight aggregate are produced specifically for skid-resistant purposes. Currently little use is being made of synthetic aggregates that have been specifically produced for use as paving-grade aggregates.

Table 5.4 has been developed to reflect the nature of the finished product, e.g., sintered, bloated, glassy, etc. The range of properties available for synthetic aggregates is much greater than for conventional aggregates,and therefore the design, manufacture, specifications, and use of synthetic aggregates must be considered separately.

5.3 Evaluation of Synthetic Aggregate Manufacture

In the following evaluation, the manufacture of various synthetic aggregates and their engineering properties will be discussed. Reference will be made to Table 5.4, in which various manufacturing methods are grouped according to the nature of the finished product. Included in the evaluation are currently produced industrial ceramics, industrial waste products, and synthetic aggregates tailor-made for use as skid-and wear-resistant paving applications.

5.3.1 Tailor Made Ceramic Aggregates--Synopal and Calcined Bauxite

Aggregates in this category are processed for use as skid resistant aggregates using high-type or specialized ceramic technology. Processed Synopal and calcined bauxite are perhaps the best known and documented of these materials. Synopal is a silicate-glass ceramic produced with a true ceraming schedule (Method 2 in Table 5.4). The raw material is first melted and then quenched to produce a granulated glass. The glass is then heated on a controlled "ceraming" schedule to nucleate and promote crystal growth. The result is a calcium silicate mineral with an amorphous glass matrix.

Its high compressive strength (6500 kg/cm^2), hardness (7.5 Mohs, about 1100 Vickers) and reflectivity under night illumination are its more favorable properties. Synopal [Dryover 1962] has been little used in the United States, principally because of its high cost ($50/ton) ($55/Mg) and marginal skid-resistance performance [James 1968]. It was, at one time, produced by the Martin Marietta Company, but it is no longer produced or marketed in the United States. Synopal has been used in test roads in Michigan [Serafin 1970], West Virginia and Illinois and in other states.

Aluminum oxide (Al_2O_3) is the most important single material in the abrasives industry. The natural forms of aluminum oxide are in reality corundum or α-alumina in varying degrees of purity and crystallization [Coes 1971]. Natural corundum, emery, and spinel have in the past been important as abrasives. Clear, colored corundum occurs as the gemstones ruby, saphire, and topaz.

For use as an abrasive, high purity alumina is formed in a fusion process where the raw materials are melted and the high purity Al_2O_3 is recovered (Bayer process). Sintering may also be employed so that finely ground alumina is extruded or pressed into shape and fired at temperatures ranging from 1450-1750 C. The firing temperature depends on the impurities in the raw material and on the details of the processing (Coes 1971]. It is important to note that the various processes for producing abrasive-grade aluminum oxide are batch-type operations which involve considerable processing of the raw bauxite to control grain size and porosity.

α-alumina may also be formed by the direct calcining of raw bauxite to a temperature of about 1500 C. Chemically combined water is lost (dehydroxylated) during the initial stages of calcining. As the calcining temperature increases, the molecular structure reorders to produce the stable α-alumina (corundum) form of Al_2O_3 [Tubey and Hosking 1972]. The products that are formed depend on the impurities in the bauxite as shown in Table 5.5. Bauxites low in quartz and kaolinite clay are required to produce a calcined bauxite high in α-alumina (corundum) and low in hematite or mullite. The term bauxite is loosely applied to a variety of rocks that contain a high percentage of hydrated aluminum oxides. The natural occurrence of bauxite is rather widespread but it is principally found in the tropics or subtropics. Major producers of bauxite are Jamaica, Surinam, Guyana, the U.S.S.R., the U.S., France and Australia. Many deposits of bauxite are economically unimportant because of impurities in the bauxite which render it unusable for abrasives or aluminum extraction.

The use of calcined bauxite as a skid-resistant aggregate has been studied extensively in Great Britain [James 1968, Tubey and Hosking 1972, Hosking and Tubey 1973]. In these studies, raw bauxite ranging up to 7 cm in diameter was calcined in the laboratory at 1500-1750 C in a rotating

Table 5.5. Minerals Which May Occur in the Calcining of Bauxite [After Tubey and Hosking, 1972]

Before Calcining	Product of Calcining
Quartz SiO_2	Cristobalite SiO_2
Kaolinite Al_2O_3 $2SiO_2$ $2H_2O$	Mullite $3Al_2O_3$ $2SiO_2$ + Cristobalite SiO_2
Gibbsite δ -Al_2O_3 $3H_2O$	
Boehmite δ -Al_2O_3 H_2O	Corundum α -Al_2O_3
Diaspore α -Al_2O_3 H_2O	
Goethite α Fe_2O_3 H_2O	
Maghemite δ -Fe_2O_3	Hematite α Fe_2O_3
Hematite α -Fe_2O_3	

drum much like a concrete mixer (about 500 lbs) (227 kg) maximum capacity. Cooling rates were essentially at furnace rate: from the soak temperature to 900 C in 30 minutes and then air cooled to ambient. The calcining (temperature) schedule was found to be critical, as might be expected. A 30-minute hold at temperature resulted in large crystals tightly bound in an unbloated glassy matrix. Too little matrix gave a friable aggregate, whereas too much sintering lowered the polished stone value.

The results of tests made on various bauxite samples calcined in the laboratory are given in Table 5.6. Obviously, calcined bauxite presents an excellent source of skid-resistant aggregate and should be pursued in the United States. Deposits of calcined bauxite exist in Arkansas and, in addition, bauxite with clays offers possibilities as a source of corundum or α-alumina.

The special qualities of calcined bauxite that make it such an excellent skid-resistant aggregate are worth special attention. Polycrystalline pure α-alumina is an anisotropic, tough, hard, very strong material as shown in Table 5.7. Differential wear or hardness is obtained by the random orientation of the grains of α-alumina which tend to retain a blocky structure as the grains wear under the action of traffic. Optimum skid resistance was obtained with 15-70 μm corundum crystals bonded by a moderate quantity of glassy matrix to yield an open texture. Water absorption of up to 5 percent was characteristic of bauxites with better polishing resistance. The high cost ($30-50/ton) ($33-55/Mg) of calcined bauxite is due to shipping costs and the high cost of the raw material. However, conventional rotary kilns should be adaptable to its manufacture.

Table 5.6. Performance and Results of Laboratory Tests [Tubey and Hosking 1972]

Description	PSV (2)	AAV (2)	AIV (2)	Grit (3) AIV	Spec. Grav.	Water absorption (%)
RASC bauxite: 1969 (1)	75	3.0	-	14	3.44	3.6
RASC bauxite: early	75	3.0	33	-	3.41	3.6
RSG bauxite: 1969 (1)	-	-	-	-	-	-
RSG bauxite: Brixton	78	-	-	16	-	-
Surinam bauxite: grey	78	3.7	-	14	-	-
Surinam bauxite: brown	64	8.2	-	27	-	-
Chinese calcined bauxite	-	-	-	-	-	-
Lafarge calcined bauxite	64	-	-	21	-	-
'Corundum' (synthetic)	-	-	-	-	-	-
Greek	62	-	-	12	-	-
Turkish emery	-	-	-	-	-	-
Dense (pelletized) bauxite	73	-	-	-	-	-
RSG Fraction A: black-balled	79	2.4	15	6.6	3.48	3.0
RSG Fraction B: black 'ring'	80	4.3	27	8.8	3.13	3.7
RSG Fraction C: plain white	64	4.4	37	11.2	2.62	3.8
RSG Fraction D: speckled white	59	3.0	35	11.0	3.61	2.1
RSG Fraction E: light brown and yellow	71	4.9	37	11.3	2.76	3.3
RSG Fraction F: dark brown and grey	79	5.3	35	-	2.82	4.3
RSG Fraction G: purple hard	59	3.7	32	-	2.54	1.1
RASC Fraction H: light blue grey -	83	-	-	-	-	-

(1) RASC - Refractory Aggregate Super-Calcined, RSG - Roadstone Grade
(2) B.S. 812, 1975 PSV = Polished Stone Value
AAV = Aggregate Abrasion Vaue
AIV = Aggregate Impact Value
(3) Modified Procedure using 3 mm grit.

Table 5.7. Properties of Polycrystalline α Alumina [Coes 1971]

Property	Value
Hardness Mohs	9
Knoop	2000
Tensile Strength	65-100,000 psi (1)
Compressive Strength	300,000 psi (1)
Modulus of Elasticity	50×10^6 psi (1)
Porosity	up to few percent
Melting Point	2040 C
Entropy of Fusion	11 calories/mole (2)
Coefficient of Thermal Expansion (Linear)	4×10^{-6} cm/cm - °C

(1) 1 psi = 6.895×10^3 pascals

(2) 1 calorie = 4.19 joules

5.3.2 Tailor-Made Aggregates--Other Possibilities

The concept of specially compounded sintered materials for use as skid-resistant synthetic aggregates has been studied in detail in a companion project [Roy 1977] and is attractive. These materials are typified by molochite which is presently produced as a refractory by firing a China clay to give mullite crystals dispersed in an amorphous phase (44 percent amorphous, 56 percent mullite), Table 5.8. Whereas molochite requires a high firing temperature and may be costly with respect to energy requirements, more cost-effective compositions may be possible with:

1. Altered raw materials giving different crystalline components in the final aggregate.

2. Neglecting the refractory requirement and thereby reducing raw material and energy costs.

3. Fluxes that reduce firing temperature.

4. Bloating to give vesicularity (improve skid resistance) and lower shipping costs per unit volume.

5. Blending hard and soft components to give a multiphase aggregate, e.g., a low temperature flux used to "cement" together a harder, high-melting temperature phase.

A more comprehensive approach to developing synthetic, tailor-made aggregates would be to study the phase diagrams which represent compositional ranges within which the most feasible ceramic aggregates could be produced; that is, cover the range of most of the abundant raw materials which would not be out of the range in cost. Figure 5.2, Na_2O-Al_2O_3-SiO_2 represents a typical clay, rich in alumina and silica, with some alkali. Fe_2O_3 could also be present as a fourth component in small quantity, which would not drastically affect the equilibria, but would lower the melting temperatures

Table 5.8. Properties of Molochite
(Data courtesy of Hamill & Gillespie, Inc., Livingston, N.J., 1976)

Indicative Chemical Analysis:	
Silica (SiO_2)	54-55%
Alumina (Al_2O_3)	42-42
Ferric Oxide (Fe_2O_3)	0.75
Titania (TiO_2)	0.08
Lime (CaO)	0.1
Magnesia (MgO)	0.1
Potash (K_2O)	1.5-2.0
Soda (Na_2O)	
Mineralogical Analysis by X-ray Diffraction	
Mullite	56%
Amorphous Silica Glass	44
Specific Gravity	2.70
Porosity*	About 6
Hardness	Between 7 and 8 Mohs scale

*Bloating can be achieved by means of a gas-generating additive in the production of "Mossite" bricks [Hosking 1974]

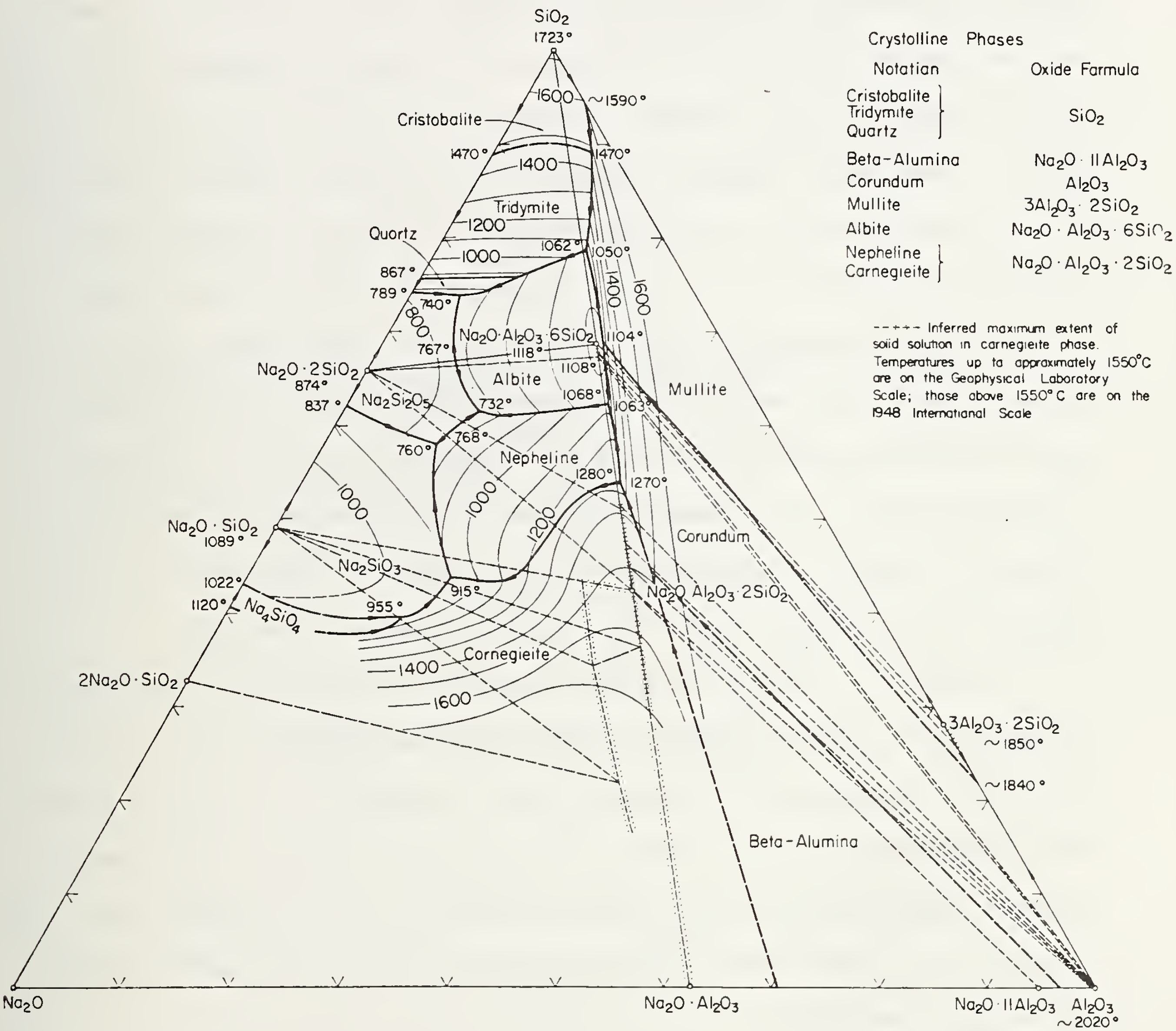


Figure 5.2. System Na_2O-Al_2O_3-SiO_2: Composite [Levin et al. 1964]

slightly. Compositions would be on the right side of the diagram (right of the line joining $Na_2O \cdot Al_2O_3$ and SiO_2) and in the primary phase field for mullite. Figure 5.3 represents the tetrahedron with Fe_2O_3 added. Figure 5.4 shows univariant and invariant equilibria in this system involving liquid. The most relevant region is represented at the top right, where tridymite (trid = SiO_2) + hematite (Fe_2O_3) + mullite ($3 Al_2O_3 . 2SiO_2$) are in equilibrium with liquid, rather than tridymite + mullite, in the simpler system of Figure 5.2.

In many clays K_2O is dominant rather than Na_2O, and the phase diagram of Figure 5.5 is applicable. Again, mullite would be the dominant phase in the typical raw material compositional regions. High-alumina clays are relatively rare; hence, the mullite-corundum mixture is less common than mullite-tridymite or mullite-glass in reaction products of calcined or sintered clays. Such compositions are characteristically poor in K_2O (or Na_2O), which is relatively advantageous, because crystal growth in feldspar fields (higher alkali) is extremely slow and impractical.

Probably the next most abundant source of raw materials is mixtures of limestone and siliceous phases, in which case the equilibria occur in the left portion of Figure 5.6 (CaO-Al_2O_3-SiO_2). Compositions near CaO and SiO_2, with the presence of a small amount of Al_2O_3 (as in clays), would lower the melting temperature to produce pseudo-wollastonite as the primary phase. The presence of a little alkali (the usual case in clays) would further lower the melting temperature.

The next figure, Figure 5.7, represents impure limestone (or dolomitic limestone)-clay (shale) mixtures. Depending upon the relative proportions, the first triangle (10 percent Al_2O_3) would represent such a composition. With a high CaO (only partially dolomitic) limestone, either pseudowollastonite or β-wollastonite could be synthesized, with a liquid

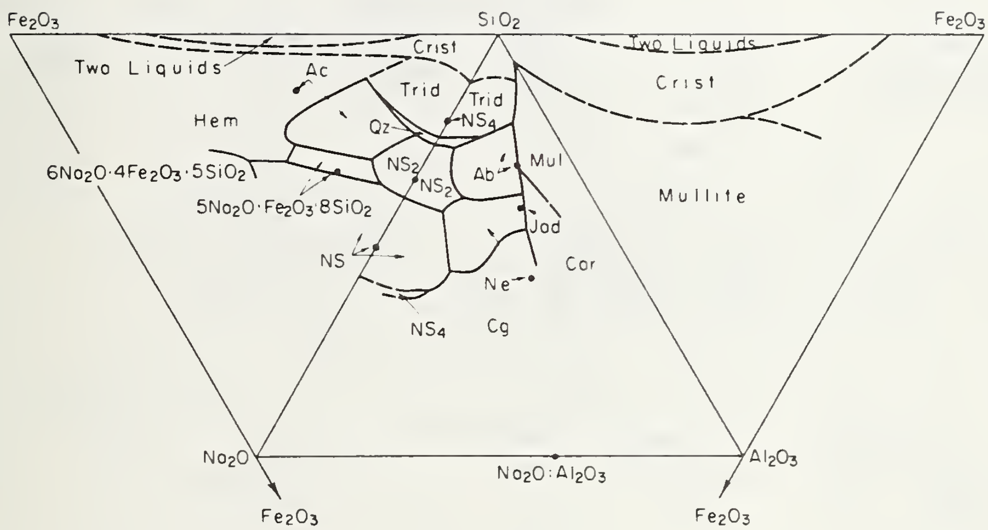


Figure 5.3. System Na_2O-Al_2O_3-Fe_2O_3-SiO_2: Quaternary, Showing Three Faces of the Tetrahedron [Levin et al. 1969]

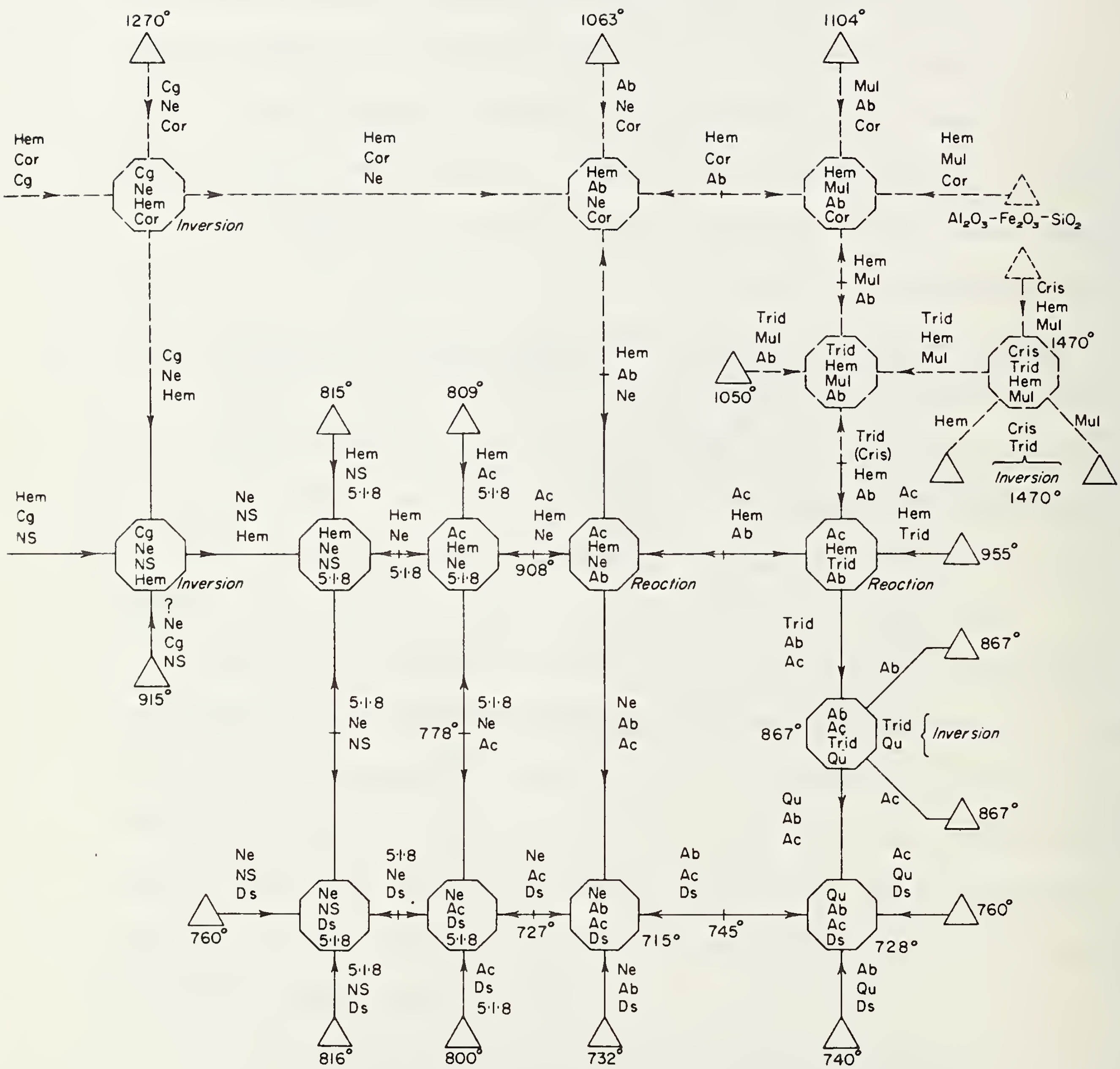


Figure 5.4. Quaternary System $Na_2O-Al_2O_3-Fe_2O_3-SiO_2$ [Levin et al. 1969]

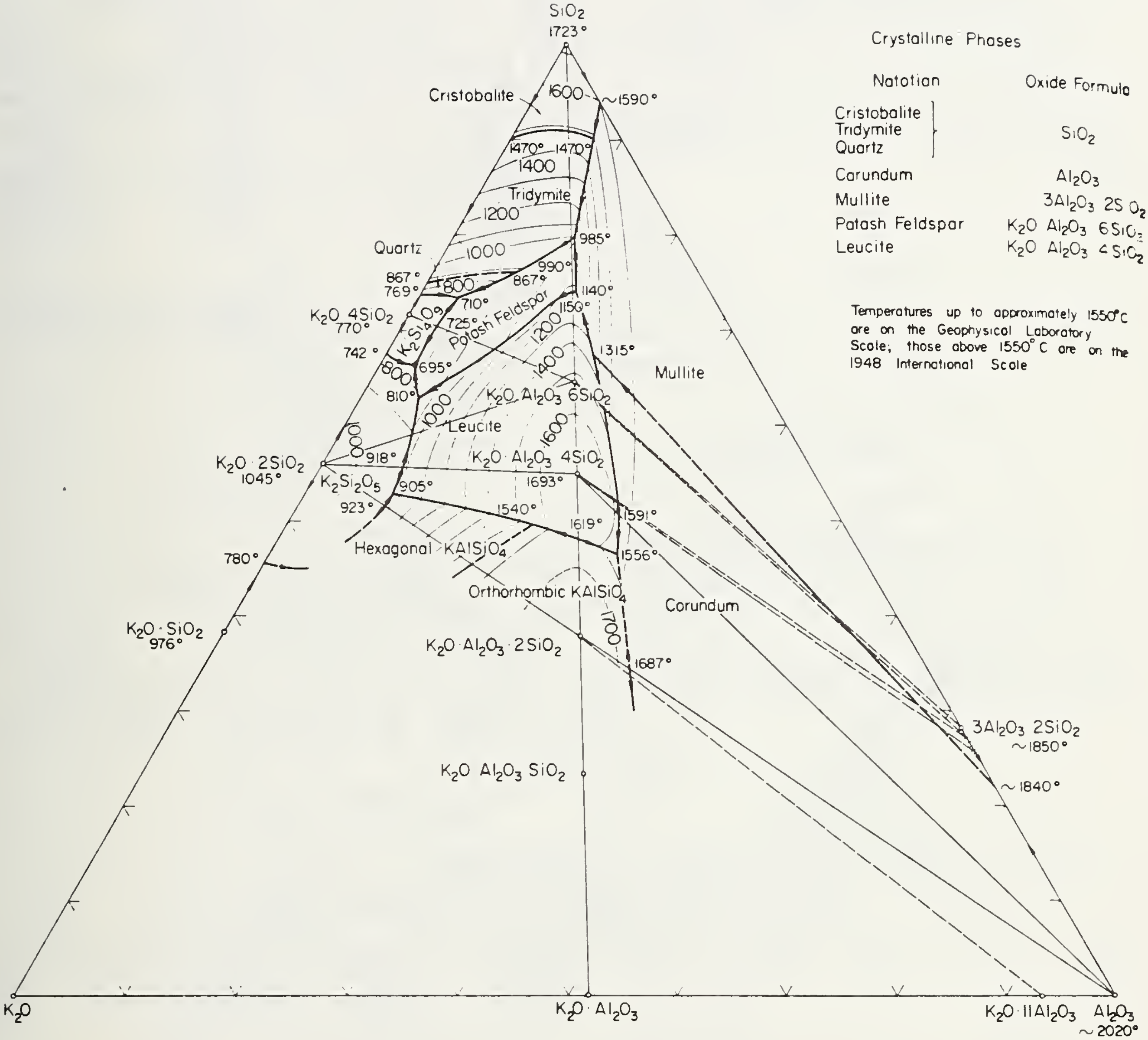


Figure 5.5. System K_2O-Al_2O_3-SiO_2 [Levin et al. 1964]

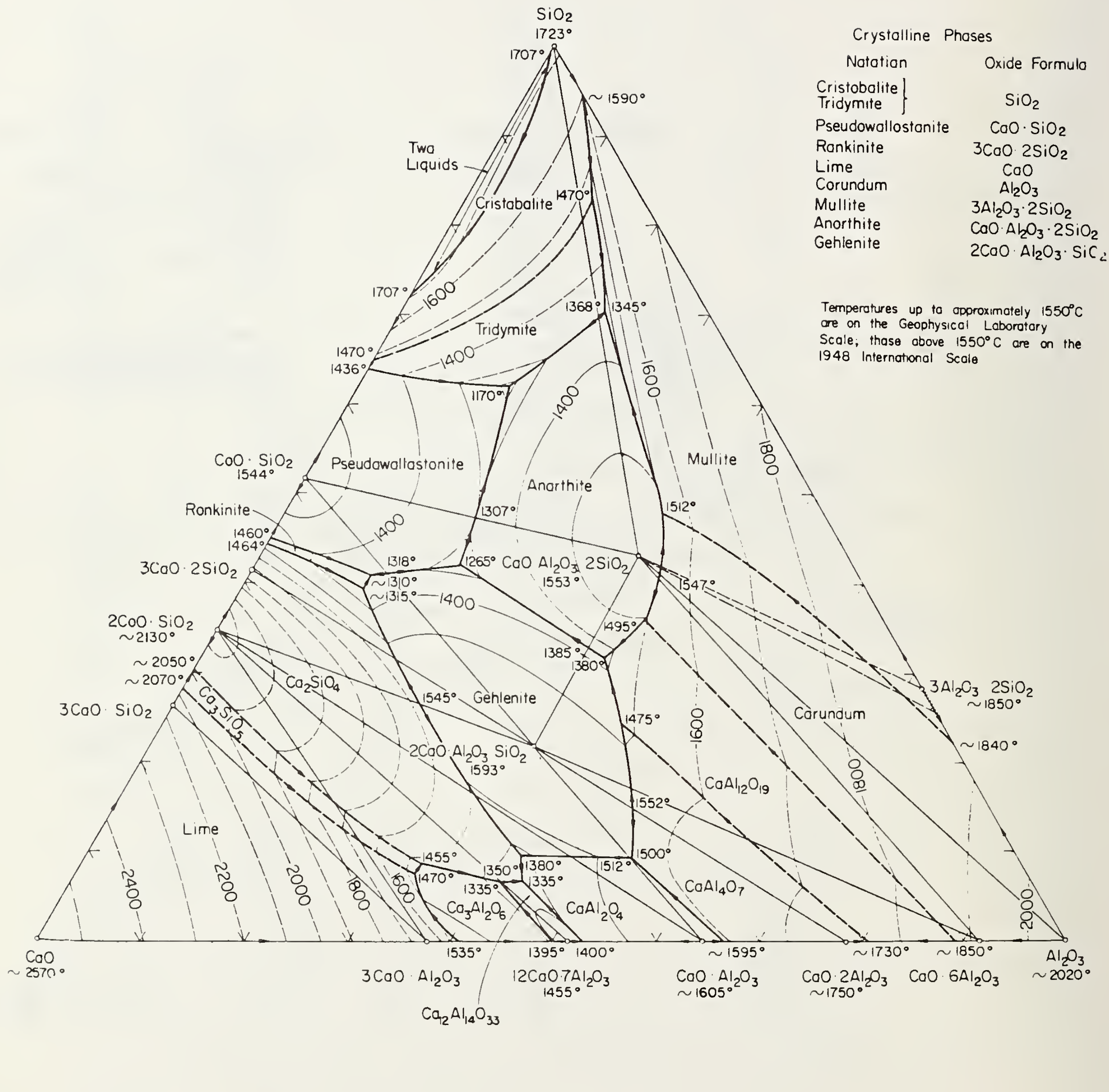


Figure 5.6. System $CaO-Al_2O_3-SiO_2$: Composite [Levin et al. 1964]

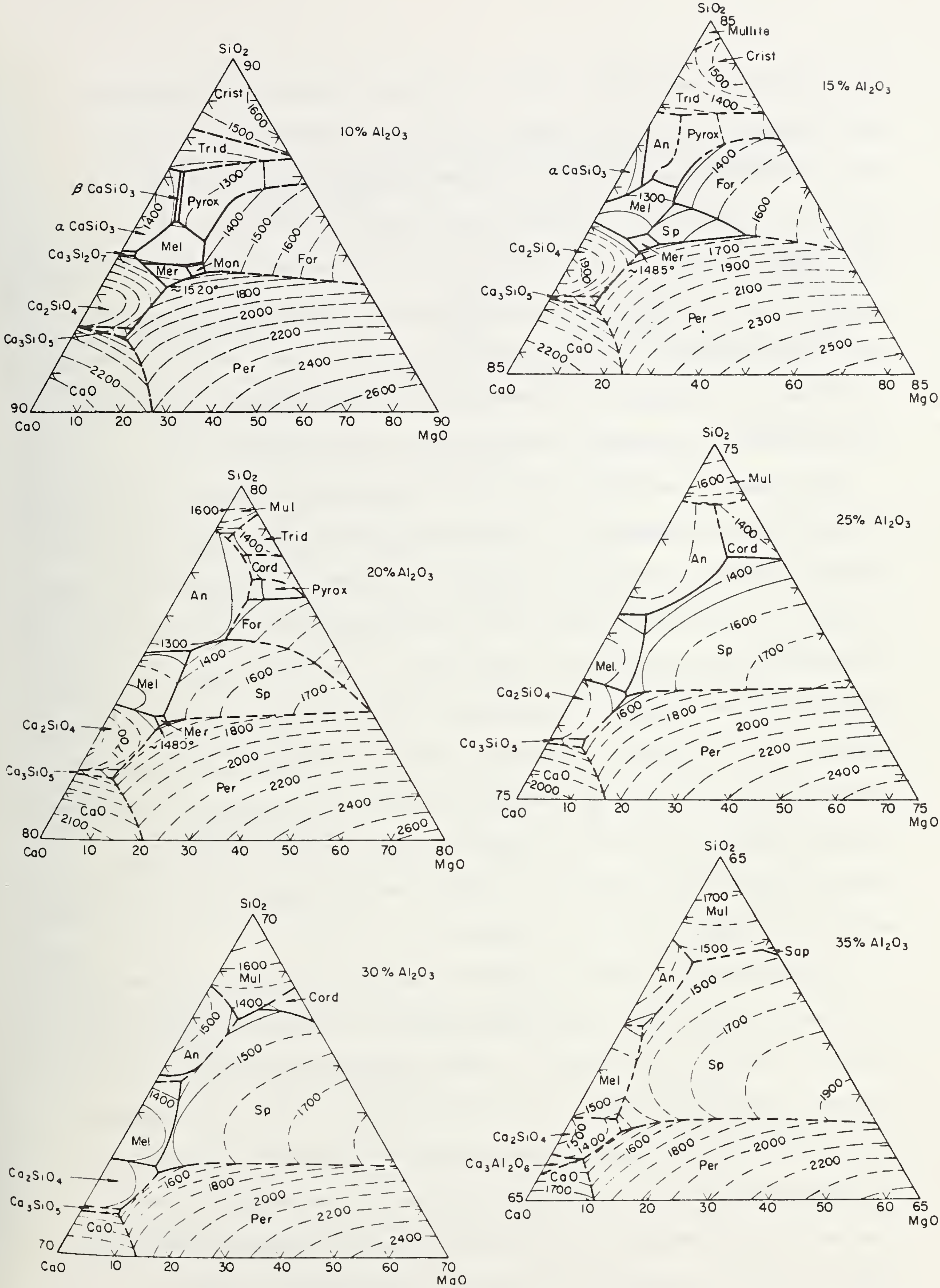


Figure 5.7. System CaO-MgO-Al_2O_3-SiO_2 [Levin et al. 1969]

phase forming below 1300°C, causing a relatively rapid sintering reaction. Alkali present would further lower the melting temperature. The compositions of blast-furnace slags also fall within this system (with the addition of iron oxide), and crystalline phases melilite, merwinite, monticellite (containing Mg, and in the first, Al) or dicalcium silicate are often present. All are quite acceptable crystalline phases although the dicalcium silicate needs to be sufficiently stabilized through solid solution so that it does not invert to the gamma form, a crystallographic transformation involving a considerable volume change, which is destructive.

In a series of studies of materials currently in production, the Transport and Road Research Laboratory has investigated various bricks for use as skid resistant aggregates [Tubey and Hosking 1974]. Some of the results are given in Table 5.9. Brick is a sintered or liquid phase sintered material which must be pressed to shape before the firing process. Generally, brick of a quality acceptable for skid-resistance purposes is refractory and hard fired, i.e., fired to a relatively high temperature allowing considerable crystal growth beyond the initial sintering. Refractory quality is not needed for aggregate use, but the hard firing may be necessary to give adequate crystalline growth for good polish resistance. Unless pressed to size, the brick would have to be crushed for use as an aggregate.

As part of a TRRL study of synthetic aggregates of controlled porosity, a refractory brick called mossite was prepared at various porosities. The results of a study of the effect of firing temperature and additive (bloating agent) content are shown in Table 5.10 [Hosking 1974]. An acceptable AAV or PSV value was attained at a porosity of 43 percent with a firing temperature of 1300°C and a 1:1 additive content. An excellent correlation was obtained between the PSV and the porosity as shown in Figure 5.8. However, some of the higher PSV values were associated with weak

Table 5.9. Results of Tests for Polishing, Crushing Strength, and Abrasion Resistance on Some Crushed Firebricks [James 1968]

Type of Brick	Polished Stone Value (PSV)	Crushing Strength (lbs/sq. in.)(1)	Aggregate Abrasion Value (lowest best) (AAV)
Silica brick (unidentified)	72	8,800	21
Scottish fireclay brick ("Nettle 1")	67	10,500	5
Scottish fireclay brick ("Dougall")	63	2,500	19
Silica brick (Pearson Kaolith 1500)	60	8,500	12
Silica brick (Morgan HT Chinaclay)	58	5,500	8
Scottish fireclay brick ("Nettle 2")	58	26,100	12
Bauxite brick (Stein 70)	83	10,000	12
Bauxite brick (Stein 73)	78	13,000	6
Bauxite brick (Stein 84P)	87	11,000	8
Bauxite brick (Alumantine 55)	84	-	36
Bauxite brick (Alumantine 63)	88	-	38
Bauxite brick (Alumantine 72)	87	-	5
Mossite brick	72	-	14
Calcined Demerara Bauxite for Comparison	75	-	4

(1) 1 psi = 6.895×10^3 pascals

Table 5.10. Results of Tests on Mossite Aggregates Produced at a Range of Firing Temperature [Hosking 1974]

Test	Firing Temperature (°C)	Proportion of Clay to Additive "C"							
		1:1	2:1	3:1	4:1	5:1	6:1	7:1	8:1
Porosity (percent)	1300	43	32	25	24	21	19	18	15
	1200	51	41	30	20	25	26	26	25
	1100	58	51	34	37	34	32	34	33
Polished-Stone Value (3)	1300	79	68	64	63	59	56	54	52
	1200	-(1)	75(1)	72	66	65	68	65	67
	1100	-(1)	-(1)	71	75	76	68	-(1)	72
Aggregate Abrasion Value	1300	12	10	7	7	8(2)	9	7	7
	1200	14	15	9	9	12	10	14(2)	13
	1100	70	-(1)	20	10	13	26	-(1)	-(1)

(1) Too weak to test

(2) Only one specimen possible

(3) Corrected to Enderby of 51

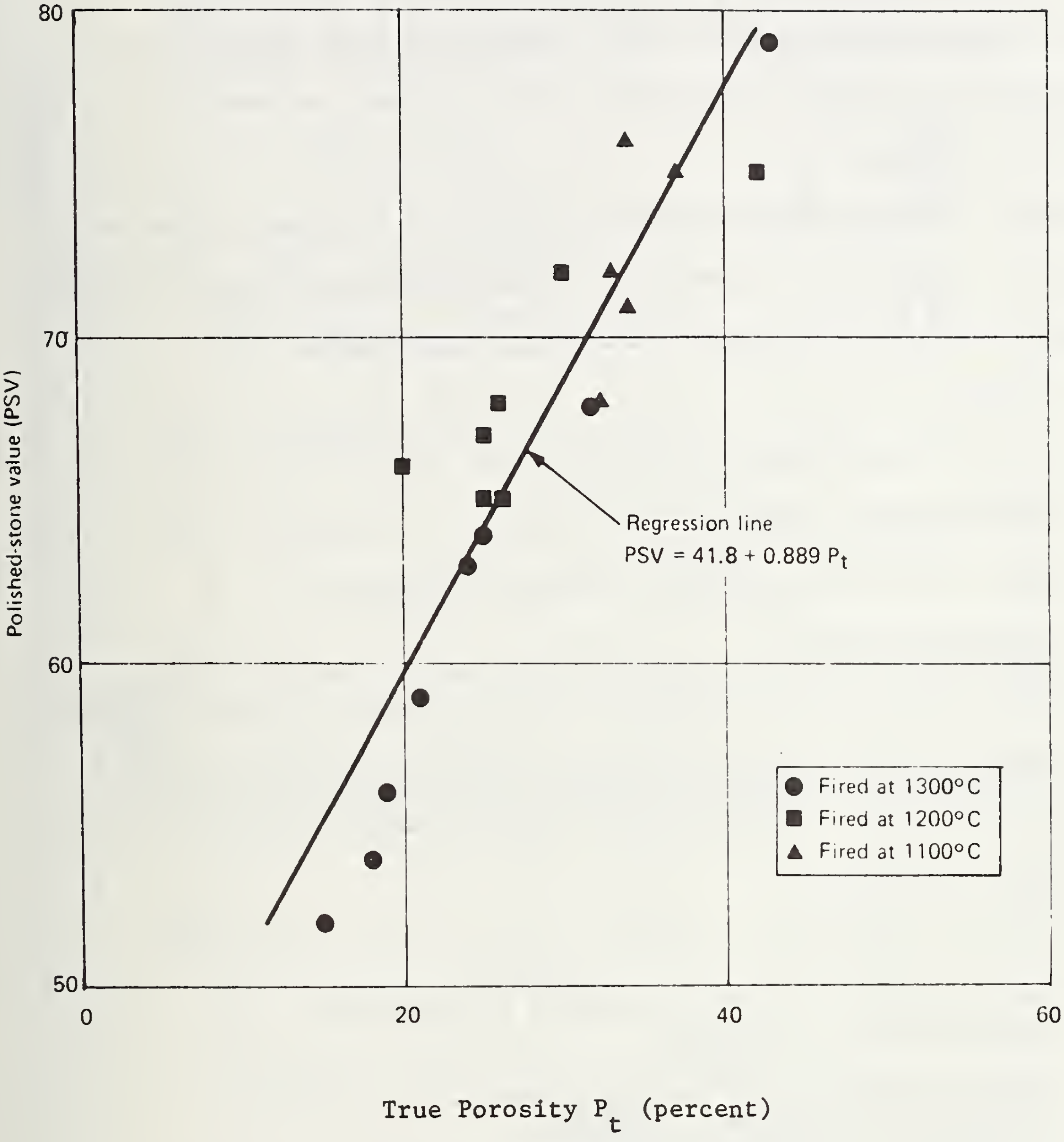


Figure 5.8. Relation Between the Porosity and PSV of Fired Molloute [Hosking 1974]

material that was unacceptable in terms of its abrasive value. It was, however, possible to produce a porous aggregate acceptable in terms of both abrasion and polishability, provided careful control was exercised over firing and composition.

A variety of other synthetic materials have been proposed for use as skid-resistant aggregates. A summary of various materials is given in Table 5.11 representing several different manufacturing processes:

1. sintering - bricks
2. sintering with bloating - mollocite
3. glass ceramics - synopal
4. glass ceramics with bloating - expanded slagceram
5. calcining - bauxite

Each one of these processes is capable of producing a skid resistant aggregate. The most desirable process depends in large measure or the nature of the particular raw materials. Manufacturing techniques must be sensitive to:

1. Energy requirements.
2. Production capacity.
3. Level of capital investment required for new plant construction or conversion.
4. Producing aggregates with multiple uses, e.g., paving aggregates and structural aggregates.

In the sections to follow, specific aggregates will be discussed; lightweight aggregates, slags, and industrial wastes. These materials all show good promise for use in skid-resistant aggregates and have the advantage of being in production with a backlog of field applications.

Table 5.11. Aggregate Characteristics for Miscellaneous Synthetic Aggregates

Material	L.A. Abrasion	Polishing Evaluation Test Method	Polishing Evaluation Test Values	AAV[4]	AIV[5]	Other[6,7]	References
Synopal	23	NCCT[1]	47	-	-	WA=2.4%	Mullen 1971
Synopal	--	PSV	50	3.5	19	S.G. = 2.60	Hosking & Tubey 1972
Calcined Bauxite RASC	--	PSV[2]	75	3.0	30	WA=3.6% S.G. = 3.41	
Calcined Bauxite RSG-F	--	PSV	79	5.3	35	WA=4.3% S.G. = 2.82	Tubey & Hosking 1972
Calcined Bauxite RSG-G	--	PSV	54	3.7	32	WA=1.1% S.G. = 2.54	
Molochite		PSV	50				Hosking & Tubey 1972
Calcined Flint	--	PSV	53	-	32	WA=1.3% S.G. = 2.25	Hosking & Tubey 1972
Crushed Flint	--	PSV	35	1	23	WA=1.0% S.G. = 2.54	
Foamed Slagceram	--	PSV	60	10	--	Heat Treated Slag	Hosking 1976
Crushed Glass	--	PSV	26	6.7	--	Not Acceptable	Hosking 1976
Fused Alumina (Abrasive)		PSV	62	2.6	--	Industrial Abrasives	Hosking 1976
Emery (Natural α - Alumina)		PSV	62-68	1.9-2.2	--	--	
Solite (Expanded Slate)	40	NCCT[1]	52	--	--	WA=3.5%	Mullen 1971
Expanded Shale	<40	RPP[3]	73	High*	--	--	
Sintered Rock Fines (one sized) I	--	PSV	72	10	--	--	
Sintered Rock Fines (graded) II	--	PSV	64	7.9		--	Hosking 1976
Fused Granite Fines (Witernite)	--	PSV	64	4.2		--	
Calcium Silicate Bricks	--	PSV	73	8.0	--	--	Hosking 1976
Refractory Brick (Modified)	--	PSV	70-75		--	Question Durability	
Sintered Red Mud	--	PSV	51	2.9	--	--	Hosking 1976
Sintered Red Mud	35	PSV	50-70*	High*		S.G. = 2.19	Hosking & Tubey 1973
Fly Ash and Magnesia	--	PSV	67	15	--	Composite Aggregates	Hosking 1976
Colliery Shale and Bauxite	--	PSV	66-70	<10	--	Composite Aggregates	
Slate/bauxite, sintered	--	PSV	70	5.7			
Blast furnace slag/calcined bauxite		PSV	63	7.3			Hosking 1970
Silt/bauxite		PSV	68-73	10-13			

*Values estimated for PSV and AAV, specific data are not available.

1 NCCT = North Carolina Circular Track
2 PSV = Polished Stone Value
3 RPP = Reciprocating Pavement Polisher
4 AAV = Aggregate Abrasion Value
5 AIV = Aggregate Impact Value
6 WA = Water Absorption
7 SG = Specific Gravity

5.4 Synthetic Aggregates Other than Tailor-Made

There are a number of possibilities for potential skid-resistant aggregates which are not specially produced as such. In some cases, these materials are currently used as aggregates (blast-furnace slag, lightweight aggregate) and in other cases they are yet to be developed as aggregate sources (taconite tailings, steel slags, pyrolysis slag). From the point of view of energy saving and environmental impact, the potential skid resistance of many of these materials should be more fully evaluated. In a number of instances, further developmental work is needed, for example, the absorptivity and wear susceptibility of sintered fly ash.

5.4.1 Lightweight Aggregate

The term lightweight aggregate includes expanded shale, slate, clay and sintered fly ash. The majority of the lightweight aggregate used in the United States is made from crushed shale fired in rotary kilns. The firing is a sintering process accompanied by bloating, the latter caused by the interlayer water released by the clay particles in the shale [Gallaway 1966]. If clay is the raw material, it must first be agglomerated by a pelletizing or extrusion process. If required, pulverized coal may be mixed with fly ash before agglomeration. The pelletized fly ash is then fired on a traveling sinter grate [Minnick 1970].

Most of the lightweight aggregate used for skid-resistance purposes has been used in Texas and Louisiana in bituminous pavements. It has been used in dense graded mixtures [Lehmann 1959], surface treatments [Maupin 1976, Gallaway 1968], and in plant-mixed seal [Adam, 1974]. In some instances, it has been used as the sole aggregate and in other instances in blends with natural aggregates.

The skid-resistant properties of lightweight aggregates are derived from their vesicular nature and their ability to maintain sharp exposed edges (cell walls) as they wear. Consequently, properly designed and constructed lightweight aggregate surfaces can maintain a high level of skid resistance throughout their service life. The use of lightweight aggregates in bituminous construction is well documented in the literature [Gallaway 1969], mainly on the basis of the Texas and Louisiana studies.

Comparison skid data for lightweight aggregate pavements are generally scattered in the literature together with data on other aggregates. Where gravels, blast-furnace slags, "trap rock" and other natural aggregates are available, lightweight aggregate is not competitive in cost. Lightweight aggregate can be justified only if it gives desired increases in skid resistance at an acceptable cost. The present cost of lightweight aggregate, \$12-15/ton (\$13-17/Mg), will undoubtedly increase markedly with restrictive uses of natural gas.

There are few reports of the use of lightweight aggregates in the northern part of the country. A Virginia study [Maupin 1976], reported a porous friction course prepared with lightweight aggregate. After three years, the skid number was 72, but traffic had been light. A comparable marble section had a skid number of 57. In an older study, Wycoff [1959] reported a 2-year-old lightweight aggregate pavement in Richmond, Virginia that had performed satisfactorily.

In view of the considerable success with lightweight aggregate in Texas and Louisiana and its limited use in northern states, more attention should be given to lightweight aggregate on a national scale. Present specifications [Ledbetter et al. 1971] (including modified specifications as used in Florida, Texas, and Louisiana) have not proven to be generally applicable to lightweight materials produced nationally. In discussing this point with a number of experts some concern has been expressed with respect to applying these specifications to new and untried sources of lightweight aggregate. Laboratory wear rates appear questionable (Figure 3.3), but these data, as well as durability, skid resistance, and construction data, need to be verified in the field.

Lightweight aggregate can also be produced by sintering fly ash [Minnick 1970]. The ash does not bloat during firing but remains at a nearly constant volume. At present, there are only two plants producing sintered fly-ash aggregate and the production from these plants are consigned to lightweight block and concrete construction. Fly-ash aggregate has a lower energy demand than expanded shale, 500 BTU/lb (1163 kJ/kg) compared to 2000-2500 BTU/lb (4650-5820 kJ/kg) for expanded shale because of the carbon content of the ash and, therefore, is potentially less expensive than expanded shale. Because it does not bloat or skin over during firing, fly-ash lightweight aggregate is very porous and potentially very absorptive of asphalt. In addition, the wall structure is very thin and, based on the visual examination of a few particles, not very promising as a skid-resistant aggregate. Samples requested from the manufacturer for polishing testing were not forthcoming. However, it is the authors' opinion that while the material may be very skid resistant it may not be very wear resistant, based on the pore structure of the material (thin walls with open porosity). Additional work would be required to improve the pore structure to increase wear resistance and reduce absorptivity. This may be a worthwhile effort in view of the hardness (7.5-8.0) of the high silica-alumina fly-ash compositions.

Another technique for producing lightweight aggregate is the sintering of coal preparation plant refuse. This material may be fine washings [Gee 1976, Myers, Pfeiffer, and Orning 1964] or the coarse refuse from the preparation plant [Rose and Decker 1977, Rose et al. 1976, Chambury et al. 1973, Utley et al. 1964]. Presently there are three plants that produce sintered refuse: Fountainlite at Pueblo, Colorado; By-Lite by the By-Lite Corporation of Pennsylvania; and a pilot plant at the University of Kentucky. One potential advantage in using this material is its fuel content. Although some supplemental fuel is required for ignition, this is less than 10-25 percent of the total energy required for sintering. Not all refuse is equally suited for sintering, mainly because of high sulfur content and related emissions problems during firing. The bloating of pure refuse is not controllable, i.e., the process is self-determining; however, the addition of clay or sand may control bloating [Rose and Decker 1977]. This would also offer the opportunity to dope the refuse with a more skid-resistant additive such as calcined bauxite.

Sintered refuse is reported by Rose and Decker as a skid-resistant and durable aggregate. The raw refuse was first hammermilled until 90 percent passed the 3/8 in. (9.5 mm) screen and then sintered on a traveling sinter grate. Test data are given in Table 5.12. Soundness values are high, 40-50 percent compared to 18 percent allowed in the ASTM specification, but reportedly this is

Table 5.12. Sintered Coal Mine Refuse Test Data
[Rose and Decker 1977]

Test	Sintered Aggregate Test Values						ASTM Specifications	TTI Recommendations[c]
	Group 1		Group 2		Group 3			
	No. 8 Grading	Open-Graded	No. 8 Grading	Open-Graded	No. 8 Grading	Open-Graded		
Soundness Loss, %								
existing[a]	41.8	48.2	46.3	42.3	52.2	49.4	18%, max.	15%, max. (can be waived)
coated[b]	4.8	7.8	4.8	7.8	4.8	7.8		
Freeze-Thaw Loss, %								
existing[a]	6.4	4.5	6.3	3.4	9.5	7.0	--	7%, max.
coated[b]	5.1	4.6	5.1	4.6	5.1	4.6		
L.A. Abrasion Loss, %	33.1		33.0		33.8		40%, max.	30-35%, max.
Dry Bulk Specific Gravity	1.38	1.43	1.36	1.47	1.38	1.44	--	--
Water Absorption @ 100 Minute, %	3.98	4.22	4.55	4.24	4.02	3.91	--	--
Saturation @ 100 Minute, %	15.9	18.5	17.4	19.7	16.0	17.6	--	15%, max (can be waived)
Asphalt Absorption, %	6.4	5.4	6.4	5.4	6.4	5.4	--	--
Dry-Loose Unit Weight lb/ft^3 (kg/m^3)	34.2 (548)	37.3 (597)	36.8 (589)	37.3 (597)	35.4 (568)	37.3 (597)	--	55%, max. (890), max.

a - Discrete aggregate particles containing no asphalt cement.
b - Discrete aggregate particles with asphalt absorption satisfied (5.90 percent by weight of aggregate).
c - Ledbetter, et al., 1971.

counteracted by asphalt absorbed into the aggregate pores, effectively sealing the pores. Asphalt absorption ranged from 5.4 to 6.4 percent and as expected, the design asphalt content for the mixtures was also high (up to 10 percent).

Frictional characteristics obtained with the ASTM E 510 small torque device are shown in Figure 5.9. Good resistance was obtained with the open graded mixture, Figure 5.9, but the open graded mixture had not yet achieved terminal polishing. Wear data are not available, and in spite of the acceptable L.A. loss, the potential wear resistance is suspect. This suspicion on the part of the authors of this report is the result of the lightweight nature of the sintered particles (<600 kg/m^3 loose unit weight) and the shortcomings of the L.A. Abrasion test, when standard and lightweight materials are compared. More attention should be given to this material, particularly if pore structure can be controlled or if other materials can be blended with the refuse to increase the wear resistance of the sintered product.

5.4.2 Blast and Steel Furnace Slags

Slags comprise a large family of non-metallic by-products from the refiring of metals from metallic ores. Blast furnace slag is the most widely used of the slags in pavement construction, especially in surfacing mixtures [Emery and Kim 1974]. The demand for blast-furnace slag from the iron and steel industry has resulted in full utilization of this material, mainly as a construction aggregate. Blast-furnace slag is also used as a constituent in the manufacture of Portland cement, and is in demand as a ballast, concrete aggregate and expanded slag. The annual world consumption of all types of blast-furnace slag is estimated at over 120 million tons (108 Tg) with nearly 30 million tons (27 Tg) produced in the United States [Rock Products 1975].

The composition of blast-furnace slag varies from furnace to furnace depending on the operating practice, raw materials, and grade of steel being produced [Lee 1975]. Fortunately, the parameters that optimize the production of steel also

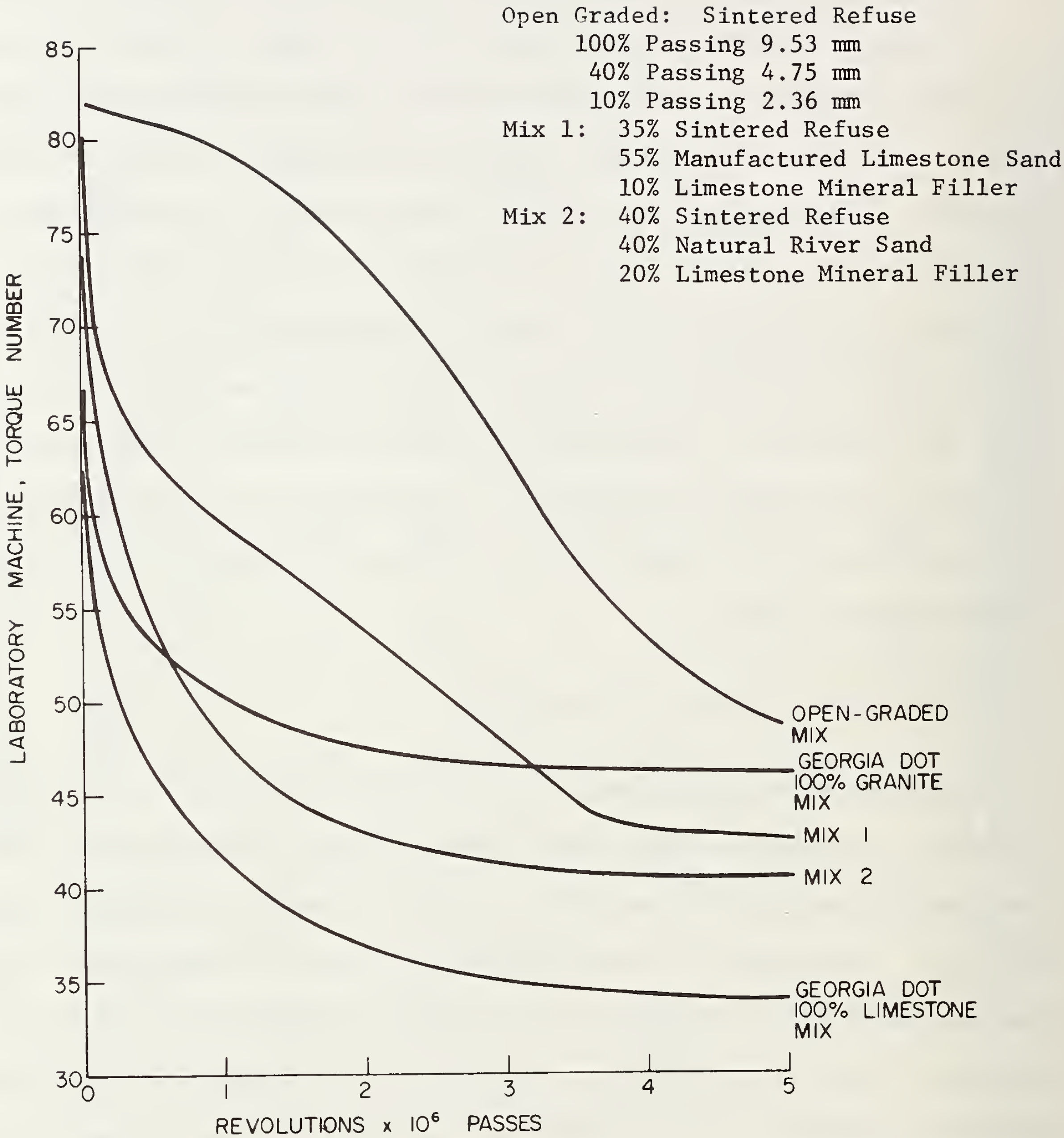


Figure 5.9. Torque Number vs. Wheel Passes
[Rose and Decker 1977]

optimize the properties of the slag for paving purposes. Average chemical composition has been given as: 36 percent SiO_2, 12 percent Al_2O_3, 42 percent CaCO, 6 percent MgO with lesser amounts of FeO, MnO, and S [Lee 1975]. Blast-furnace slags are alkaline in nature.

Mineralogical composition is controlled by rate of cooling and chemical composition; with a more rapid rate of cooling and a higher silica content giving finer crystal size [Josephson et al. 1949]. Water-quenched or granulated slag is amorphous and is preferred in cement manufacture. Air-cooled slag is crystalline and is preferred for use as an aggregate in Portland cement concrete and bituminous concrete pavement construction.

Blast-furnace slag may also be expanded with steam or a jet of water. A cost premium of \$3-4/ton (\$3.3-4.4/Mg) must be paid for expanded slag. Given the high level of performance of air-cooled slag, there is little reason to specify expanded slag in skid-resistance application, except perhaps for lowered shipping costs per unit area of in-place pavement. Expanded slag is used principally in block construction and lightweight concrete.

The skid resistance of blast-furnace slag in general is well accepted and documented, and it is used by many states as a preferred skid-resistant aggregate in bituminous surface mixtures. For example, blast-furnace slag is now shipped from the Birmingham, Alabama area to Florida for use in surface mixtures.

There are few references in the literature to the exact mechanism by which slags develop their skid-resistant properties. Indeed, not all slags are equally effective as skid-resistant materials [Hegmon undated]. Undoubtedly, the vesicular and hard nature of the slags plays a major role, but the presence of differential wear is less established.

An English study examined the mineralogical and physiographic properties that influenced the Polished Stone Value (PSV) of blast-furnace slag [Gutt and Hinkins 1972]. They found that the slag was heterogeneous with different compounds in the slag having different PSV. Increasing porosity and crystalline size improved the PSV of the slag, but large porosities gave poor wear resistance. Increased crystalline size did not alter the wear resistance.

Steel slag is quite different. Steel slag has a high unit weight, 120 lbs/ft^3 (1920 kg/m^3) loose, and is potentially expansive. Bituminous mixtures made with steel slags averaged 180-190 lbs/ft^3 (2900-3000 kg/m^3) [Emery and Kim 1974]. The slag must be aged in stockpiles to allow hydration of the calcium and magnesium oxides if they are to be used in a confined situation or in an alkaline environment [Crawford and Burn 1969]. Steel slags may be treated with spent pickling liquors to stabilize expansion. Steel slag contains a considerable quantity of iron and may be recycled in a blast furnace for additional iron recovery.

Steel slag may be from an open hearth or a basic oxygen unit; the latter is expected to account for more production in future years. Annual production of steel slags (approximately 10 million tons (9 Tg)) in the United States is less than half that of air-cooled blast-furnace slag (26 million tons (24 Tg)), with a per ton value about half that of air-cooled blast-furnace slag (the latter about $2.00-4.00/ton ($2.2-4.4/Mg)).

Properties of steel slag are quite varied, because of plant-to-plant variation and within-plant variation. This variation can become an important factor in the utilization of the material [Personal Communication, Garland Steele, W. Va. DOH, 1976], affecting expansion potential, mixture design, and skid resistance potential. Steel slags can give surfaces with excellent skid-resistance and should be more fully utilized.

There is little reported research on the skid resistance of steel slags per se; most of the data on steel slags is contained within reports on several materials [Bransford 1971]. The use of a Canadian steel slag in a number of bituminous applications has been reported in detail [Emery et al. 1973]. Uses included: 3/4 in. (2 cm) precoated "chippings" surface dressing, slurry seals, and dense graded hot mixtures. Reported Marshall stabilities run very high, in excess of 4,000 pounds (1814 kg), with VMA and air voids in an acceptable range.

The advantages and disadvantages of blast-furnace and steel slags have been listed by Corkill [1975]. For blast-furnace slag the advantages are:

1. Maintains good skid resistance.
2. Non-polishing.
3. Provide great heat retention aiding placement and compaction.
4. Low unit weight ($\simeq$80 lbs./cu. ft.) (1230 kg/m^3).
5. Non-expansive nature.
6. Uniformity of material.
7. Stable and inert.
8. Able to obtain high Marshall stabilities.

The disadvantages in using the blast furnace slag are as follows:

1. Not a hard aggregate like steel slag or traprock.
2. Highly absorbent nature soaks up moisture and can create problems drying at the plant.
3. Increased asphalt cement content required because of absorption.

When used in bituminous concrete surfacing, Corkill [1975] reports that steel slags are:

1. Highly skid resistant.
2. Resistant to wear.
3. Resistant to stripping of asphalt cement because of presence of free lime.
4. Provide great heat retention aiding placement and compaction.

5. Able to obtain very high Marshall stabilities.

6. Rough textured and very hard.

The disadvantages of using steel slag are as follows:

1. High unit weight (~180lbs./cu. ft. (2880 kg/m^3)

2. High transportation costs of the heavier material.

3. Steel slag particles have a tendency to swell and pop out of the surface.

4. Questionable availability of supply.

Steel and blast-furnace slags require further research, especially the steel slags. Not all blast-furnace slags give good skid resistance,and the factors that affect the skid resistance of blast-furnace slag warrant further investigation. A summary of data from various sources is given in Table 5.13. Crystallinity, porosity or surface texture, differential hardness, and crystallographic orientation with respect to differential wear are some of the factors that should be considered.

The mechanism by which steel slags acquire their skid resistance needs to be defined. Processing these slags to produce a lightweight aggregate should be investigated,so that they may be more economical (shipping costs) for other than local use. The influence of production variations and stockpiling or aging procedures must also be evaluated.

5.4.3 Industrial By-Products

A number of industrial by-products have been proposed as alternate sources of aggregate [Miller and Collins 1974]. Before these varied materials can be used as surfacing aggregates, each has to be reviewed on its own merits. Most of the viable candidates are either the result of the crushing of rock, mine tailings, or the result of refining or incineration. Examples are incinerator refuse [Pindzola and Collins 1975],

Table 5.13. Polished Stone Values for Selected Slags

Material	Polishing Evaluation Test Method	Polishing Evaluation Test Values	AAV[2)]	Other[3)]	
Corby Slag (Blast Furnace)	PSV[1)]	49	9	S.G. = 2.74 WA=0.8	Hosking & Tubey 1972
Steel Slag	PSV	57	-	-	
Steel Slags	PSV	44-67	3		
Blast-Furnace Slag	PSV	40-63	7-29	S.G. = 2.49 WA=2.0	Hosking 1976
Aluminum Slag	PSV	35-38			
Blast-Furnace Slag	PSV	52	8.5		Gutt 1972
As Above w/ Heat Treatment	PSV	62	7.7		
Slagceram foamed/processed	PSV	69	6.2	-	
Foamed Slagceram	PSV	60	10	-	Hosking 1976
Modified Blast-Furnace Slag	PSV	69	6.2	-	

1) PSV = Polished Stone Value
2) AAV = Aggregate Abrasion Value
3) WA = Water Absorption
SG = Specific Gravity

smelter waste [Hughes 1973], spent oil shale [Gromko 1975], and metallurgical slags [Emery 1975]. Many of the industrial slags are glassy while others are at least partially crystalline. The glassy slags, in spite of their hardness, are poor candidates for skid-resistant materials. Increased vesicularity may improve their skid resistance properties. As a result of being quenched in water, many are relatively small sized (passing the 2.36 mm or No. 8 mesh sieve). They often suffer in toughness because of their brittle, glass-like characteristics, and in the pavement tend to produce concoidal fracture planes parallel to the pavement surface, resulting in loss in texture and frictional resistance.

The glassy slags are hard and might serve as excellent aggregates, if they could be processed to produce crystalline rather than glassy particles or if they could be expanded to improve their texture. Both heat treatment and fluxing would probably be required. Additional research is needed on the processing of (molten) industrial slags to improve their properties if they are to become strong candidates as skid-resistant aggregates. Bloating procedures and/or thermal processing with fluxes to give compositions that crystallize more readily would appear to be viable research approaches. The limited availability of these slags may preclude any significant expenditure of research and development funding.

Municipal incinerator residue from several cities has been considered as an aggregate for asphaltic wearing courses. The material is heterogeneous, containing ash, metal fragments, crushed glass, etc. [Haynes and Ledbetter 1975, Collins 1976]. These materials do not show much promise for wearing courses, although they may find use as a binder or base material.

A method of producing a "bubble aggregate" has been reported [Aitcin and Poulin 1974]. Briefly, a waste organic or a non-sintering mineral waste is pelletized and then coated with a clay or other flux. The result is an expanded bubble with many of the properties of an ordinary lightweight aggregate. While the process is perhaps not yet proven for general materials, the concept should be pursued as a means of agglomerating industrial dusts that are not otherwise utilizable. The concept has the potential for controlling differential hardness and, hence, skid resistance.

A composite aggregate has been made on a laboratory scale by fusing a silicious mining waste with a lower melting temperature glass. This multiphase aggregate gave good skid resistance after polishing on a circular test track [Ramsay and Davis 1975].

The coal-burning power plant is a major source of industrial waste. Two types of waste are produced: fine fly ash (minus #200 mesh) and coarse bottom ash or boiler slag (plus #200 mesh). In order to be used as a surfacing aggregate, the fly ash must be agglomerated, either by thermal or chemical means. Sintered lightweight fly-ash aggregate is discussed under the heading of lightweight aggregate.

A binder must be added to fly ash in order to agglomerate it chemically. Portland cement and sulfate waste, for example, have been used, but the end product is of questionable potential as a skid-resistant aggregate [Smith and Larew 1975]. The high-lime, lignite fly ashes are potentially much more reactive than the eastern bituminous fly ashes and, perhaps, they should be evaluated in light of ceramic technology with the idea of producing an aggregate by hydrothermal means. With some additional chemical or hydrothermal treatment, some fly ash could potentially produce an aggregate with favorable skid-resistant properties, especially if the aggregate were doped with hard particles to give differential wear.

Boiler slag is produced by water quenching molten slag that is drawn from a slag tap furnace. The slag is-one sized, typically #16-#8 mesh, angular, glassy, and lacking in microtexture. In this regard, it is similar to many other glassy industrial slags.

Boiler slag has been promoted as a skid-resistant aggregate under the trade name Black Beauty. It is no longer in widespread use as a skid-resistant aggregate, and there are varied reports as to its effectiveness. Prior use has been in sand mixes, slurry seals, and surface treatments, but its use is infrequently reported. Skid data on field sections paved with various boiler-slag mixtures do exist in various Highway Departments but are too often considered privileged information. In one confidential report, an SN of 48 and 42 at 40 and 60 mph (64 and 97 km/h) respectively was cited for a 1-year-old heavily travelled boiler-slag surface. Open graded boiler-slag mixes with initial BPN Numbers of 80-90 are possible with boiler slag, but their skid resistance and durability under continued or heavy traffic use is open to question [Usmen and Anderson 1977].

Bottom ash results from the burning of pulverized coal over open grates and is collected at the bottom of the power plant stack. Current annual production is about 12 million tons/year (11 Tg/year). It may be produced in various forms, as a fine sand, agglomerated or poorly sintered fly-ash particles, and may even contain some slag-like particles. Most bottom ashes have a range of sizes from the #200 mesh (75 μm) to 1-in. (25.4 mm).

Bottom ash has not been used as a surfacing aggregate except as reported in West Virginia [Usmen and Anderson 1977]. Although mixtures can be made with acceptable Marshall or Hveem properties, bottom ashes generally do not have the toughness required of a surfacing aggregate. When used as the sole aggregate, they break down under the action of traffic to form a fine-textured surface lacking in macrotexture. Bottom ash may have potential as the fine aggregate in surfacing mixtures, if it is crushed before use to control the breakdown [Anderson 1977].

A host of other mining wastes are potential candidates. These materials have been reviewed [Collins and Miller]. These materials are considered outside the scope of this study; however, they should not be over looked in the future. Many of the mining wastes are merely waste rock and can be treated as conventional aggregates. In other instances (e.g., uranium tailings), there are serious environmental concerns to be considered.

5.5 Evaluation of Aggregate Beneficiation

Aggregates requiring beneficiation may be divided into two groupings: those that possess adequate durability but inadequate skid resistance and conversely those aggregates that possess adequate skid resistance but inadequate durability. Beneficiation should therefore be approached from two viewpoints: 1) improving durability, and 2) improving skid resistance. Both approaches may be valid routes for offering skid resistant aggregates [Marek, 1972].

Heat treatment is a potential method for the beneficiation of skid resistance, but the unprocessed aggregate must be such that either a phase change or crystal growth occurs on heating [Roy 1977]. Examples are synopal [Dryover 1962] (crystal growth), slagceram [Davies, et al., 1970] (crystal growth) and calcined bauxite [James 1968] (crystal change). Thermal treatment has been covered earlier in Chapter 5, albeit, not under the heading of beneficiation. Industrial slags and waste materials may be improved by heat treatment or by the injection of steam or air as with expanded blast furnace slags. Additional modifications could be made by adding fluxes or harder mineral phases to the molten slag.

Another approach to beneficiation for skid resistance is the addition of a hard coating to a softer aggregate. For example, corundum might be added to a soft limestone. To be successful, the added material would most likely have to be crystalline; a low temperature glassy coating would be too soft. As mentioned earlier in Chapter 5, flame spraying does not seem to be viable from an economic standpoint. Thick coatings or more correctly, composite aggregates, do appear promising and have been discussed previously. Denis and Massieu [1975] prepared a polymer-sand mortar having superior properties with various sand-size and smaller fraction materials, using only a 10 percent epoxy. This may have potential for aggregate, but would be expensive.

The use of coatings and penetrants appears promising for improving aggregates that are deficient in durability or that exhibit excessive asphalt absorption. Several organic coatings have been evaluated using absorptive limestone [Dutt and Lee 1971] and both the water absorption and asphalt content were lowered. The cost effectiveness of the particular combinations reported [Dutt and Lee 1971] might be questioned, but there may well be instances where polymer impregnation might substantially improve durability. For example, a gneiss, one in particular reported as very skid resistant by the Connecticut Department of Transportation [personal discussion with ConnDOT personnel], might show adequate durability if impregnated with poly methyl methacrylate. Coatings and penetrants may be used to improve freeze-thaw resistance, mechanical strength, bonding strength, and/or water absorption. Studies relative to the upgrading of aggregates using chemical coatings are under way at The Pennsylvania State University (NCHRP project 4-12, Upgrading of poor or marginal aggregates) and at the Brookhaven National Laboratory (FHWA agreement P.O. 7-3-0070, Improvement of wear resistant properties of aggregates by materials impregnation).

It has been recognized for many years that blending is an effective method for improving the skid resistance of poorly performing aggregates. Laboratory and field experimental data have shown that the skid resistance of blended aggregates is improved in proportion to the percentage of hard and soft constituent aggregate [Gramling and Hopkins 1974, Dahir and Meyer 1974, Mullen and Dahir 1974]. The Texas State Department of Highways and Public Transportation has developed a test method to evaluate aggregate blends and an equation to predict the skid resistance of blended aggregates [Texas Test Method Tex-438-A, Rev: May 1976]. Accordingly, the percentage by volume of non-polishing aggregate to use in a blend to meet a specified Polish Value is determined by the following formula:

$$\%P_R = \frac{100\ (PV_S - PV+2)}{PV_R - PV}$$

$\%P_R$ = Percent (by volume) of the non-polishing coarse aggregate

PV_S = Polish Value required by specification

PV_R = Polish Value of non-polishing aggregate

PV = Polish Value of polish aggregate to be improved

However, there appears to be a range over which the effect of blending is most effective 50-70 percent by volume of the good aggregate [Gramling and Hopkins 1974]. On the other hand, smaller amounts of high performance synthetic aggregate could conceivably be used if methods are found to expose these aggregates to the tire contact. This is an area that appears to merit further research. Other areas that should be investigated are (a) the possibility that a blend may perform as well or better than the higher performing ingredient in the blend [Dahir and Meyer 1974] and (b) the effect of utilizing larger size particles of the better performing aggregate in a bituminous surface mixture [Mahone 1974].

5.6 Summary

A wide variety of approaches are possible for the production of synthetic or man-made skid-resistant aggregates. Manufacturing techniques, raw materials, and aggregate properties are so interrelated that one cannot be specified at the exclusion of the other. A scheme for selecting or producing new synthetic aggregates might be:

1. Determine chemical composition of raw material to evaluate potential for growth of hard crystals, e.g., potential for producing α - (alumina), Section 5.3.2.

2. Consider modifying the raw material by fluxing, doping with harder material (e.g., doping with calcined bauxite) or other means.

3. Determine optimal aggregate type for particular material as in Figure 3 (e.g., bloated sintered, Table 5.4).

4. Select manufacturing process compatible with material and aggregate type (e.g., sintering), Table 5.4.

5. Perform economic analysis of proposed system considering energy costs, capital outlay, market area, production capacity, etc.

6. Produce material; bench scale and pilot plant scale.

7. Perform laboratory wear and polish tests along with other conventional aggregate tests.

8. Perform field demonstration study based on pilot plant material.

9. Release to market place for commercial development.

One of the more troublesome aspects of the above outline is the lack of test methods for evaluating polishing and wear. The British Test methods are the most rational but experience with these methods has been mostly with conventional aggregates, or aggregates which behave similar to conventional aggregates, so that extrapolation to more exotic aggregate types is risky. For example, blast-furnace slag often shows better in the field than in the laboratory [Emery 1973]. Clearly, more work is needed in the development and verification of laboratory polishing and wear test methods (especially wear).

Sintered materials such as clay, shale, and slate appear the most viable in the short run. More use could be made of non-bloating shales and clays, particularly if they could be doped with hard, fine materials such as crushed bauxite or quartz sand. Their wear resistance and applicability to northern climates needs to be verified.

Alumina based materials such as calcined bauxite, sintered red muds, and high alumina clays appear promising. A detailed search for raw materials and a laboratory scale development program for these materials should be initiated to promote commercial development.

Based on the British literature, refractory brick materials appear promising. Current methods of brick manufacture are inefficient (require crushing and sizing), and developmental work should be initiated to determine ways to efficiently transfer brick manufacture technology to the manufacture of aggregates.

Beneficiation should also be pursued. At the same time, the applicability of current material specifications may need to be reviewed relative to surface aggregates; clearly there are many aggregates with marginal durability, but excellent skid resistance (e.g., gneiss and many porous sandstones). Impregnation of polymers or inorganic cements should be evaluated with the idea of improving both the environmental and mechanical durability of these materials. Beneficiation for the purpose of increasing skid resistance by thermal treatment or coatings does not appear to be of high priority from an economic standpoint.

6.0 TECHNICAL EVALUATION OF PAVEMENT SURFACING SYSTEMS

6.1 State-of-the-Art Review and Analysis

The literature on pavement surfacing is extensive, reflecting a long-time concern of agencies and investigators who are involved and/or interested in providing long-lasting, safe and economical surfaces. Long-lasting surfaces must be resistant to all forms of wear and disintegration, whereas safe surfaces must provide adequate skid resistance and hydroplaning resistance under the most potentially critical conditions of the particular surface. Both wear resistance and skid resistance, together with the other surface parameters discussed earlier, must be provided at a cost that the agency responsible for providing and maintaining the surface can afford. Accordingly, viable alternatives are normally considered, and the most economical among them is usually applied.

A review of the literature reveals that the great majority of pavement surfaces that have been widely used to date fall basically into two broad categories: Portland cement concrete (or rigid) surfaces and bituminous (or flexible) surfaces [Yoder 1975]. The quality of PCC surfaces is highly dependent on the strength and texturing of the surface mortar, which contains 40-50 percent fine aggregate [PCA 1968]. Texturing of the surface is performed while it is in the plastic state, and grooving may also be undertaken if it becomes necessary at a later date. The top coarse aggregate layer (1 to 1-1/2 in. or 25-40 mm) just below the mortar surface is also important, if heavy traffic uses the surface and could wear the mortar in less than, say, 20 years. Other PCC surfacing systems to date, conventional and innovative, have basically consisted of procedures to improve either the quality (strength and polish resistance) of the mortar or to improve the texturing or grooving of the surface. In the case of bituminous surfacing, the emphasis has been on three aspects: (a) the quality, mainly wear resistance and polish resistance, of the aggregate, particularly the coarse aggregate when used; (b) the long-lasting adhesive quality of the binder, i.e., not allowing it to strip off the aggregate or to bleed or flush into the tire contact area; and (c) the method(s) of providing a surface texture that will have adequate, long-lasting friction and fast water drainage in the tire-contact area. Bituminous surface texturing has generally been provided either by varying the aggregate particle size, shape, and gradation, or by sprinkling the surface with high-quality aggregate having sharp particles ranging in size from 3/4 to 1/8 in. (19 to 3 mm), and precoated with an adequate adhesive that will assist to keep them in the surface under the whipping action of traffic. Innovations have varied from using high-quality, high-cost synthetic or natural aggregate (e.g., calcined bauxite and emery), to using admixtures like epoxy and shredded rubber to improve the binder, to surface grooving which has not generally been successful in rejuvenating bituminous surfaces for a reasonable length of time.

In locations where extremely high wear resistance and skid resistance are critical, and where interruptions to traffic for maintenance are costly and highly inconvenient (as in the case of toll booth approaches on high speed highways, sharp curves, stopping or lane change sections and busy ramps and bridge decks), both the aggregate and the binder may need to possess higher qualities than conventional bituminous mixtures do. This is a case where expensive innovations, as the epoxy-calcined bauxite applications, have been used, but the usage is limited to relatively small sections and few locations.

Many variations in both PCC and bituminous surfacing systems have been tried, and some innovative procedures have been introduced. Some have brought about improvements, while others have not been very successful for various reasons. In a recent survey [NCHRP 1976], ten systems, which are generally known to paving agencies, were compared relative to their wear resistance, skid resistance, cost range, and estimated life. These systems and comparisons are included in Table 6.1. Other concepts have been suggested and some have been implemented on an experimental basis. One promising concept relative to PCC surfaces is to construct the top 2-in. (50-mm) layer of a PCC pavement using high quality aggregate (both coarse and fine). Another concept is to use a 2-in. (50-mm) bonded concrete overlay. This concept has been tested in three separate projects in Iowa [Knutson 1977], but the results are not yet known.

Obviously, in any conventional or innovative pavement surfacing system, use of high quality, polish-resistant, and wear-resistant aggregates in conjunction with high quality binder and good texture will result in an optimum surface. Since bituminous adhesives and polymers bind the aggregates physically, most known high quality aggregates, synthetic and natural, are compatible with bituminous and epoxy binders. Exceptions may occur when the aggregate is hydrophillic, as is the case with some gravels. In these cases, compatible asphalt and/or anti-strip chemicals mixed with the bituminous binder normally correct the problem. When the aggregate is to be used in chemically-reacting binders as in Portland cement, it is imperative that compatibility of binder and aggregate is verified to avoid mixtures that may develop harmful reactions after the surface has been in service.

Table 6.1. Comparative Properties of Wear- and Skid-Resistant Systems for Immediate Implementation, Used as Remedial Treatments [NCHRP 1976]

	SYSTEM									
Property	PCC, Optimum Design	Dense-Gr. AC, Opt. Design	Open-Graded AC	Skip-Graded AC	AC with Precoated Chips	Epoxy-Modified AC	Asphalt Seal Coat	Rub. Asph. Seal Coat	Epo.-Asph. Calc.-Baux. Seal	Sawed Long. Grooves
Applied Cost Range ($/sq. yard)[1,2]	NA	0.75-1.50	0.50-1.20	1.00-1.75	2.00-2.50	6.75-11.25	0.20-0.30	0.50-0.75	6.00-10.00	0.50-1.00
Estimated Effective Life (Years)[2]	5-15	5-15	5-10	5-15	10-15	>10	2-5	2-7	7-13	4-7
Relative Wear Factor[2,3]	3	3	4	2	1	2	6	5	1	4
Relative Resistance to Hydroplaning[4]	4	4	1	4	3	4	2	2	2	2
Estimated Skid Number Range (SN_{40}-ASTM Trailer)[2,4]	50-65	50-65	55-65	50-65	55-70	55-65	50-65	50-60	65-80	40-50
Relative Skid Speed Gradient[5]	3	3	1	3	2	3	2	2	1	3

[1] In terms of usual resurfacing thicknesses for plant-mixed and seal coat systems and of cost experience with grooving. 1 sq. yd. = 0.84 sq. m.

[2] Estimated assuming similar aggregates (except for the system where calcined-bauxite only is specified), with effective life varying with conditions such as traffic density, climate, etc.

[3] Wear in wheel track due to abrasive action of normal traffic (1=most resistant to wear).

[4] Ratings are on the basis of "1" being most desirable.

[5] Indicates fall-off of skid number with increasing speed (1 = least change).

6.2 Summary of Selected Systems

Of the surfacing systems listed in Table 6.1, those that have been most commonly and successfully used to date for long-lasting, high performance, and may be improved as the quality of aggregate and binder improves, are the following systems:

1. Dense-graded asphalt concrete using polish-resistant, wear-resistant aggregates.
2. Open-graded asphalt concrete, hot plant mix using polish-resistant, wear-resistant aggregates.
3. Epoxy-asphalt seal coat using high quality aggregate (e.g., calcined bauxite).
4. Portland cement concrete, high strength design, using polish-resistant, wear-resistant aggregates and texturing with metal tines or combs.
5. Sawed transverse or longitudinal grooving of PCC surfaces.

Systems 1 through 3 may be applied to newly constructed surfaces or to remedy worn and/or polished surfaces of either bituminous or PCC pavements. System 4 is recommended only for new pavements, and system 5 is recommended only as a remedial procedure for PCC surfaces.

These systems have been described and evaluated in the literature in varying degrees of detail. One recent work [NCHRP 1974] includes a description of each of these and other systems; therefore, there is no need to repeat the descriptions. However, it should be added that some recent improvements on the systems were not included in the NCHRP descriptions. For example, the use of metal tines or combs rather than wire broom for texturing PCC, was not discussed.

6.3 Prefabricated Pavements

Some high cost surface treatment systems used at critical locations such as intersections may be suitable for factory prefabrication. Systems using expensive high grade aggregates and binders such as calcined bauxite and epoxy resin binders have proven effective for treatment of critical locations [James 1971 and Page 1977]. These systems might find wider use in prefabricated form since they not only utilize high cost materials, but special equipment and trained personnel are required for application. Two-component epoxy binders must be mixed in the proper ratio to a close degree of tolerance to achieve maximum durability, and curing times are necessary, during which the roadway must be closed to traffic. Since the aggregates are expensive, any excess application must be minimized or recovered. These systems are usually applied to small areas and are not practical when the projects are separated by large distances necessitating the dispatch of equipment and crews to isolated projects. Many of these objections could be overcome by manufacturing prefabricated sections of these binder-aggregate systems under controlled conditions. A suitable adhesive to bond the sections to the pavement might be preapplied, as in the case of prefabricated, pavement-marking materials. Also, suitable epoxy or thermoplastic adhesives could be used. Such an approach would enable high-performance surfaces to be shipped to potential users for application as needed. A small application was performed in 1969 by the author, using a square foot prefabricated tile and a thermoplastic adhesive. The material adhered well but the idea was not reported in the literature.

Only one prefabricated pavement system was found in the literature [Bense 1975, Bense 1976, and Bisson et al. 1976]. This system, the "Nancy Carpet," was developed in France and has performed well in preliminary tests. Full-scale production was scheduled for late 1977. The Nancy Carpet is produced in roll form and is bonded to the road surface with a thermally activated adhesive. The material consists of a thick,compliant backing material on which the binder and aggregate are mounted. The flexible support provided by the backing reduces the vulnerability of the aggregate to studded tires. A machine for application of the system has been developed which heats the adhesive, deploys the carpet, and rolls over it to press it onto the pavement. Traffic can use the pavement immediately after installation. Although not mentioned in the references, another attractive feature which might be possible is the development of a carpet-removal system which would precede the applicator in a resurfacing operation.

Portable,aluminum,aircraft-landing mats (MX-19 Landing Mats) were developed by the U.S. Air Force for constructing temporary airstrips in areas such as Southeast Asia. These were precoated with two-part epoxy onto which various types of abrasive particles were sprinkled. Modified pavement paint machines [Wald 1967] were developed for field applications of the epoxy-abrasive system to replace worn or stripped-off coatings. Since this system was developed for a specific purpose, it is unlikely to be suitable for highway applications.

7.0 ECONOMIC ANALYSIS OF RESURFACING SYSTEMS

The objective of this economic analysis is to identify the resurfacing systems that are most economical for a given set of conditions. Several resurfacing systems that are likely to have high skid and wear resistance have already been mentioned in this report. In order to provide a framework to compare these and other systems which may eventually be developed, a cost-benefit type of approach will be presented. The cost-benefit procedure will enable the user to estimate the costs of providing different aggregate and pavement surface properties and to measure the benefits associated with those properties.

Any attempt at a detailed evaluation of the total costs and benefits of a particular resurfacing system would probably prove to be extremely cumbersome. Fortunately, a knowledge of total costs and benefits is not necessary for an economic comparison of two alternative resurfacing systems. What is necessary is a knowledge of the cost items that differ in each of the two systems and the net benefits of one system over the other. Thus, there is no need to know the total cost of an item, as for example the total cost of maintenance, if the maintenance requirements for two systems are exactly the same. Even if the maintenance requirements are not the same, all that needs to be established is the difference in requirements and the difference in costs.

The economic analysis will thus be concerned with incremental costs and benefits rather than total costs and benefits. Incremental cost-benefit ratios will be computed for the various resurfacing systems. These ratios will be useful in eliminating the more undesirable alternatives, thereby giving the decision-maker a shorter list from which to choose. Non-economic factors will then enter into the decision process, making the list even shorter.

The problem of creating a general model to estimate the cost and benefits of all possible candidate resurfacing systems is a complex one. Certainly, the estimation of initial costs should at least take into account the cost of raw materials, manufacturing facilities, transportation, and construction. On the other hand, the tangible benefits associated with alternative aggregate and pavement surface properties should include items such as reductions in motor vehicle accidents, surface maintenance, energy consumption, and noise pollution, to name but a few.

Benefits can essentially be classified into three categories: those which can be valued in dollar terms (e.g., reductions in costs), those which can be quantified but are not easily valued (e.g., reductions in travel time), and those which defy either quantification or valuation (e.g., increases in creation of comforts or conveniences). For the economic analysis to be realistic, all of these benefits must somehow be considered.

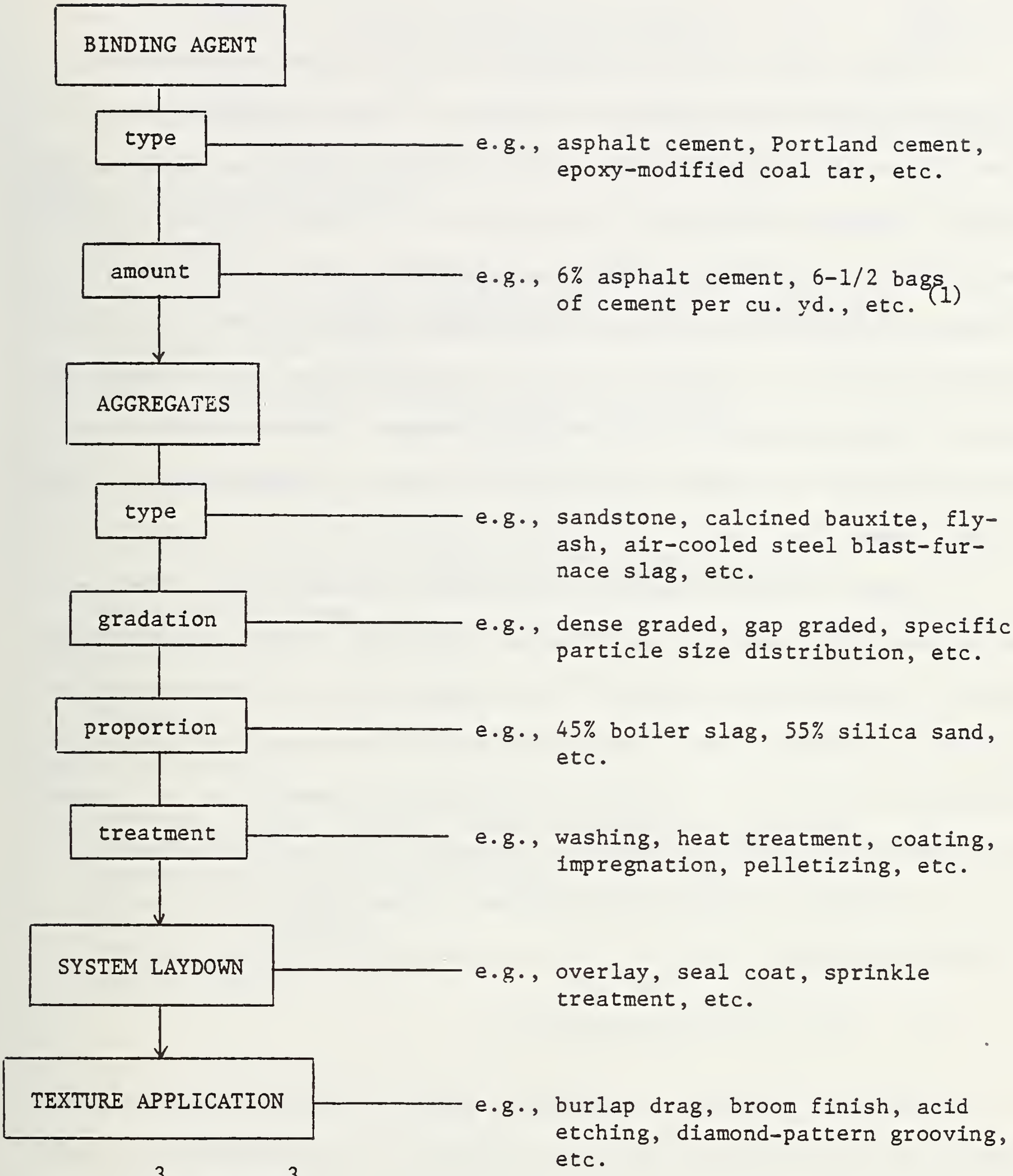
The approach which will be taken here consists of 1) the identification and discussion of all cost and benefit items which may be necessary in comparing two alternative resurfacing systems, 2) a cost-benefit evaluation of several field-proven, conventional resurfacing systems, and 3) a sensitivity analysis to determine what effect a change in assumed values has on the final results. The evaluation of conventional systems will provide guidelines for evaluating additional systems which may be developed in the future. While a lack of field data on new systems appears to be a serious difficulty, it is likely that any new resurfacing system which is introduced will basically be a conventional system incorporating one or two departures or innovations. It is therefore hoped that the evaluation of the costs and benefits of several conventional systems will enable a subsequent evaluation of the innovative and future resurfacing systems simply by adjusting the appropriate cost or benefit component.

7.1 Method of Classification

As data on various aggregates and surfaces were being collected in this research, it became obvious that a method was needed for classifying the many resurfacing systems which are presently available as well as those systems which show promise and may eventually be utilized. The method of classification that was developed is shown in Figure 7.1.

This method is aimed primarily at assisting a cost-benefit study by simplifying the comparison of specific resurfacing systems. With this method, a resurfacing system must be described according to the type and amount of binder; the type, gradation, proportion, and treatment of aggregate(s); the means of laydown; and the type of texture application (if any). This description of a proposed resurfacing system will make possible the quick computation of its costs and benefits by allowing the proposed system to be referenced to a more conventional system, the costs and benefits of which are already known.

According to this method of classification, an epoxy-modified asphalt seal coat with calcined bauxite chips, for instance, is a different system than an epoxy-modified asphalt seal coat with gravel aggregate. The only difference between the two systems, however, is the difference in the costs and benefits associated with the use of the two different aggregates. If the costs and benefits of one of the systems are known, the costs and benefits of the remaining system can readily be determined.



(1) $1 \text{ yd}^3 = 0.76 \text{ m}^3$

Figure 7.1. Method of Systems Classification for Cost-Benefit Study

7.2 Costs

The costs that will be investigated in this section are the costs associated with the initial capital investment. It may be possible, in the case of some of the more commonly used resurfacing systems, to obtain an immediate estimate of this initial cost (such as in dollars per square yard). However, when an immediate estimate is not possible, the initial cost can be broken down into the cost of the constituent materials that are to be incorporated into the system, the cost of combining these constituents, the cost of transporting them or the completed mixture. These individual cost items can then be estimated and combined to obtain an estimate of the initial costs.

In order to provide guidance in estimating the cost items, these can be further broken down into cost elements, which may be as detailed as necessary. An example of a system of cost elements which has been devised to yield the estimated,initial capital investment cost is shown in Table 7.1. Referring to Table 7.1, it should be noted that the quantification of every cost element will seldom, if ever, be necessary. The main use of Table 7.1 is as a checklist to compare the difference in the initial costs of two resurfacing systems.

7.2.1 Cost of Constituent Materials

As indicated in Table 7.1, the major elements that influence the final cost of the constituent materials are the costs of the raw materials, their processing costs, and their transportation costs. Because the cost of transportation is a variable which can significantly affect the delivered cost of a material, it will be discussed separately; thus, the costs that will now be presented are the costs of the processed material at the point of production.

Table 7.1. Cost Elements for Estimating Initial Cost of Resurfacing System

<u>Cost of Constituent Materials</u>

Raw materials
Processing (crushing, washing, heavy media separation, impregnating, coating, blending, heat treating, pelletizing, fusion, agglomeration, sintering, hydrothermal hardening, plastic binding, etc.)
Storage
Transporting (loading, hauling loaded, unloading, returning empty)

<u>Cost of Mix Production</u>

Equipment (clamshells, aggregate storage and feeder bins, elevators, conveyors, driers, dust collectors, screens, banks, scales, etc.)
Moving plant to site
Assembling plant
Operating plant
Proportioning
Hauling mix

Cost of Construction

Equipment (pavers, screeds, rollers, vibrators, supportive equipment, etc.)
Preparing surface (wetting, tack coat, fill cracks, etc.)
Storage forms, steel, etc.
Finishing or grooving
Treating (curing, waterproofing, winter protection, etc.)
Quality control

Even when the point of production is used as the common basis for comparison, the cost of processed material is still a variable dependent on a number of factors. These factors include the inherent value of the material, the market demand, and other possible uses for the material. Competitive market conditions in a particular area will ordinarily dictate the price range for a material to be used in highway construction.

Table 7.2 gives price ranges for a number of materials which may be considered as constituent material when incorporated in a resurfacing system. The prices shown are those for October 1976. They were obtained from tabular price listings found in Engineering News-Record, as well as from telephone interviews with a representative sample of producers. Since prices vary from year to year, the figures in Table 7.2 should only be used as a relative guide. The price range for each material in Table 7.2 reflects both regional price differences and physical differences in the material. Thus, the price of a given material for a geographic area which is known to have high materials prices is likely to be at the higher end of the range. Similarly, the price of a one-sized aggregate, for example, or an aggregate having a gradation not commonly produced by a quarry, would also tend toward the higher end of the range.

7.2.2 Cost of Transportation

Since the transportation costs of low value bulk materials, such as most aggregates, comprise a large percentage of the total product costs, it is necessary to discuss the transportation costs in as much detail as other production costs. The total cost of transportation can be thought of as being composed of two separate cost categories the fixed cost of loading and unloading and the variable cost of transporting the material. The unit loading and unloading cost remains the same for any one particular set of

Table 7.2. Representative Prices of Constituent Materials (Price at Point of Production)

Material	\$/ton[1]	\$/gal.[2]	Remarks
Crushed Stone	1.80-3.50		
Gravel	1.60-4.00		
Sand	1.70-4.00		
Boiler slag	1.50-3.00		
Blast-furnace slag	2.00-4.00		
Metallurgical slag	1.50-3.00		
Expanded slag	4.00-8.00		
Expanded shale	12.00-15.00		
Expanded slate	12.00-15.00		
Expanded clay	12.00-15.00		
Expanded glass (Synopal)	42.00-55.00		
Crushed brick	20.00-40.00		
Calcined bauxite	30.00-50.00		
Asphalt cement	50.00-70.00		
MC asphalt cutback	70.00-90.00		
Asphalt emulsion (RS1)	60.00-90.00		
Tar	65.00-90.00		
Sulfur	35.00-45.00		
Portland cement	30.00-50.00		
Epoxy resin		58.00-70.00	10 gallon cans
Air entraining agent		1.00-1.30	bulk
Water reducer		3.50-4.00	bulk
Retarder		3.70-4.25	bulk
Accelerator		1.15-1.35	bulk
Lime	50.00-90.00		
Pozzolan	4.00-6.00		
Curing compound (White Membrane)		3.50-4.50	55 gallon drums
Steel rebar	220.00-380.00		

(1) 1 ton = 907 Kg
(2) 1 gal. = 3.8 liters

conditions while the transporting cost varies with the distance. Thus the total cost of transportation can be seen to be a function of quantity and distance.

Truck, rail, and barge are the three principal means by which constituent materials are transported. In the U.S., the majority of conventional aggregates are transported by truck. According to a study by the Bureau of Mines [1975], nearly 90 percent of the sand and gravel and 70 percent of the crushed stone is trucked to market. Also, most such aggregates are hauled to a market which is less than 15 miles (24 km) from the plant.

For these truck carriers, haulage rates are calculated by a variety of methods. Some companies charge \$.60/first ton-mile (\$.41/first Mg-km) and \$15/add. ton-mile (\$.10/add. Mg-km). Other companies use zones, and any aggregate hauled into a given zone will cost a set price. Still other producers prefer to sell all their aggregates at a set price and manipulate the haulage price. The nature of the haul often influences the cost of transportation. Hauling down the valley costs considerably less than trucking over the mountains. Also, open-highway driving is cheaper than driving through densely populated areas.

The price of hauling aggregate by truck is often quoted by carriers as \$.15/ ton-mile (\$.10/Mg-km). Actually, the rate varies inversely with the distance. On the average, to truck aggregate will cost a customer about \$.60/ton-mile (\$.41/Mg-km) for one mile (1.6 km); \$.17/ton-mile (\$.12/Mg-km) to haul five miles (8 km); \$.10/ton-mile (\$.07/Mg-km) for 12 miles (19 km); \$.08/ton-mile (\$.055/Mg-km) for 20 miles (32 km); and \$.06/ton-mile (\$.04/Mg-km) for 40 miles (64 km). The price of hauling most other constituents or of hauling the completed mix is usually more expensive, since special purpose trucks often are employed. For example, the cost of hauling Portland cement concrete is \$1.25/yd^3-mile (\$1.00/m^3-km) for the first mile (1.6 km); the cost for 12 miles (19 km) is about \$.27/yd^3-mile (\$.22/m^3-km).

Since trucking charges may account for 50 percent or more of the total cost of delivered aggregate, transportation costs are often responsible for limiting the use of many aggregates in highway construction. When stone, sand, and gravel producers exist together, for instance the deciding factor as to which one is used, other than project specifications, is its proximity to the construction project. The distance beyond which it becomes uneconomical to haul conventional aggregates is usually quoted at 50 miles (80 km). The added benefits of using special aggregates which are not available locally must outweigh the additional costs of transporting these aggregates. Ideally, the comparative cost of utilizing special aggregates in a given situation should be determined by conducting a detailed marketing analysis in the area involved.

7.2.3 Cost of Mix Production

The cost of mix production is essentially the cost of combining the constituent materials into a material (or mix) which is to be placed on the road surface. For many resurfacing systems, it is only necessary to combine the constituent materials at the point of placement. An example is a seal coat, in which a binder is first applied to the road surface and then aggregate deposited on top. For such systems, the costs associated with a stationary batching plant can be disregarded. However, for the majority of overlays the cost of mix production can be significant. The two major mix types are Portland cement concrete and bituminous concrete. Using conventional aggregates, representative costs for these completed mixes are given in Table 7.3. For unconventional mix types, the prices in Table 7.3 can be adjusted according to changes and/or additions in constituent materials or required plant equipment.

Table 7.3. Representative Costs for Major Mix Types
(Price at Point of Production)

Portland Cement Concrete

$23 to $31 per cu. yd.[1] for 5 bag mix
$28 to $36 per cu. yd. for 6-1/2 bag mix (typical paving mix)
$45 to $54 per cu. yd. for high early strength mix (open to traffic in one to three days)

Note: 1 cu. yd. ≅ 3800-4000 lbs.[2]

Bituminous Concrete

$12 to $15 per ton[3] for dense graded wearing course with 6% asphalt cement

1. 1 cu. yd. = 0.76 cu. m.

2. 1 lb. = 0.45 kg

3. 1 ton = 907 kg

7.2.4 Cost of Construction

The cost of construction, as has been used here, is the net cost to place the resurfacing system on the roadway. Several published guides are available for estimating the construction cost elements. Dodge [1976] and Means [1976] are two of the most popular. They provide a yearly breakdown of construction cost elements into their material, equipment, and labor cost components.

When the cost of construction is added to the cost of the completed mix, the resulting sum is the total initial cost of the resurfacing system. For conventional resurfacing systems, where a broad base of historical data is available, it has already been stated that estimates of the initial cost are often available. In many cases, these initial costs are quoted in dollars per square yard. For innovative systems, the initial cost can best be estimated by employing the cost elements, i.e., adding or subtracting cost elements from a comparable conventional system.

There are a number of factors which affect the construction cost of a resurfacing system. These factors include the size of the job, the size of the crew, the type of equipment, weather conditions, etc. When these factors are considered along with the variable materials costs, any estimate of the total initial cost of a resurfacing system will at best be a range of values. Table 7.4 gives the estimated range of initial costs for a few,selected conventional resurfacing systems. These are the systems which may be used as standards for initial cost comparisons.

Table 7.4. Initial Cost Estimates for a Few, Selected Conventional Resurfacing Systems

	$/sq. yd.
Continuously Reinforced Portland Cement Concrete Overlay (5 in. thick)	$6.00 to 8.00/sq. yd.
Dense Graded Bituminous Concrete Overlay (1-1/2 in. thick)	$1.00 to $1.75/sq. yd.
Open Graded Bituminous Concrete Overlay (3/4 in. thick)	$0.60 to $1.00/sq. yd.
Gap Graded Bituminous Concrete Overlay (1-1/2 in. thick)	$1.25 to $2.00/sq. yd.
Bituminous Concrete Overlay with Sprinkle Treatment (1-1/2 in. thick overlay)	$2.50 to 3.20/sq. yd.
Epoxy-Modified Bituminous Concrete Overlay	$7.00 to $12.00/sq. yd.
Asphalt Seal Coat	$0.25 to $0.50/sq. yd.
Asphalt-Rubber Seal Coat	$0.80 to $1.10/sq. yd.
Epoxy-Asphalt, Calcined-Bauxite Chip Seal Coat	$6.50 to $11.00/sq. yd.
Sawed Longitudinal Grooves	$0.70 to $1.20/sq. yd.

Note 1. The above estimates assume the use of locally available, conventional skid-resistant aggregates (except for that resurfacing system which specifies calcined bauxite).

Note 2. 1 sq. yd. = 0.84 sq. m; 1 in. = 25.4 mm.

7.3 Benefits

In this study, a cost is a direct outlay of cash for an investment. A benefit, on the other hand, is a favorable consequence resulting from that investment. A benefit can also be thought of as a saving in costs; however, it is often difficult to value benefits in dollar terms. A benefit can be either positive or negative, depending upon whether the consequence is desirable or not.

In the evaluation of alternative resurfacing systems, one is faced with two basic questions related to benefits. What benefits should be included in the analysis, and how should these benefits be evaluated? Technically, any benefit which results from the use of a particular resurfacing system should be considered in this analysis (that is to say, many other benefits should be considered besides those strictly associated with skid resistance and durability). For example, if a new resurfacing system that showed promise from a skid-resistance and durability standpoint also possessed thermal properties such that it would remain free of ice and snow when other roads were covered, then this advantage should obviously be regarded as a benefit because it would result in decreased winter maintenance costs. From a practical point of view, however, the possible benefits are so numerous (especially when considering potential resurfacing systems) that any benefit model selected for economic analysis at this point must give attention only to those benefits which are the most common and significant. For this reason, only those benefits popularly associated with conventional resurfacing strategies are discussed here. With the development of new resurfacing strategies, additional benefits which are considered significant can also be included in the analysis.

Once the decision is made as to which benefits are to be included, it is necessary to establish a method for evaluating them. For each alternative resurfacing strategy, the relative impact of the benefits must be predicted and, where possible, quantified. Ideally, the benefit should be estimated from some known property of the resurfacing system and then converted to a dollar value. However, significant benefits that cannot be put into monetary terms should not be disregarded. These benefits can be listed qualitatively, as items of additional consideration to be weighted by the decision-maker.

The magnitude of the benefits, like that of the costs, is dependent upon the specific situation. However, because of widely varying situations, the expected ranges of the benefits of a given resurfacing system are too large to be of use. Therefore, in this study, the benefits will be evaluated by examining several representative traffic situations (i.e., situations assuming a given average daily traffic (ADT), speed, etc.).

The benefits which are considered for incorporation in this economic analysis are reductions in accidents, noise, maintenance, vehicle operating costs, and several others. These benefits will now be discussed.

7.3.1 Accidents

Skid resistance, as measured by the skid number (SN), is probably the most logical property to use for predicting the number of accidents on a given pavement surface. Few isolated studies have been made, however, which permit conclusions to be drawn regarding this relationship. This is because the relationship is frequently obscured by the presence of other variables which influence the number of accidents, such as road geometry, ADT, degree of visibility, etc. In discussing any relationship between SN and accidents, it is therefore necessary to define the conditions under which the relationship exists.

A study by Rizenbergs et al. [1974] interprets accident data in such a way that accident occurrence appears to decrease rapidly with increasing SN, up to an SN_{40} of about 40 or 45, then continues to decrease, but more gradually, for SN_{40} above 45 (see Figure 7.2). The Rizenbergs study was conducted on rural, four-lane, controlled-access highways on the interstate and parkway systems in Kentucky; only reported accidents were analyzed. The expression of accident occurrence that was found to correlate best with skid resistance was wet-weather accidents per 100 million vehicle miles (1 mi=1.61 km) travelled under all conditions. Since other studies have shown that a very small percentage of the total number of accidents in dry conditions involves skidding [Giles and Sabey 1959, Mills 1959], it will be assumed here that the number of accidents in dry conditions will not be affected by changes in SN.

In order to help define the relationship between accident rate and skid resistance for conditions other than those of the Rizenbergs study, the following information is provided:

1. Speed does not contribute heavily to accident rates. It is the differences between the speeds of vehicles in a traffic stream which contributes heavily to the accident rate. Speed, of course, does contribute heavily to the severity (i.e., cost) of accidents [Solomon 1964, Prisk 1959].

2. Traffic accident rates are normally believed to be proportional to the volume of traffic on the facility--the greater the traffic volume, the higher the accident rate--within the capacity of the highway. For traffic volumes greater than 7000 ADT, however, a decrease in accident rate is in evidence for further increases in ADT [Kihlberg and Tharp 1968 Belmont 1953, Schoppert 1957].

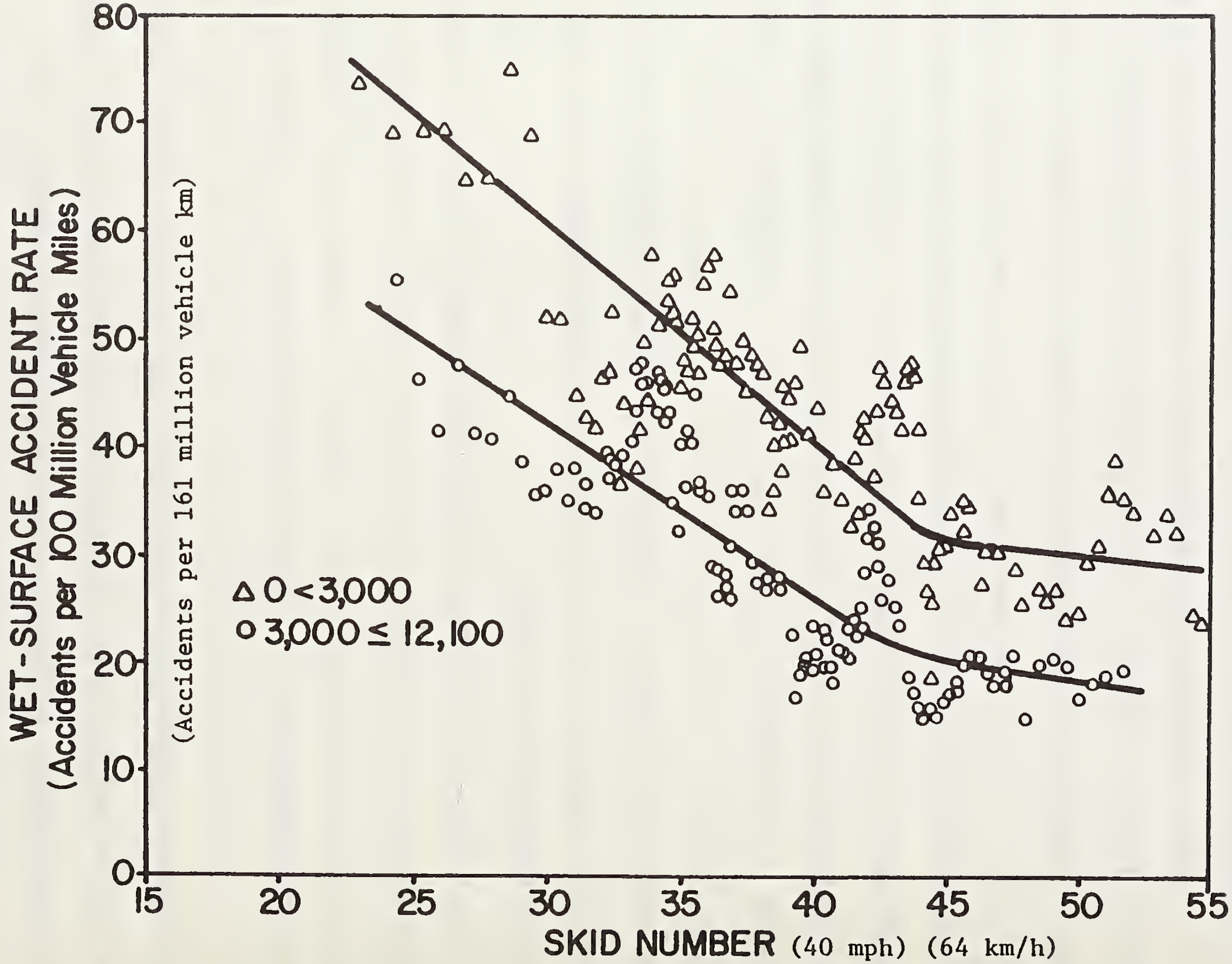


Figure 7.2. Accident Rate vs. Skid Number
[Rizenbergs et al. 1974]

3. The number of lanes appears to have no relationship to the accident rate, i.e., multilane highways appear to have accident rates comparable to those on two-lane highways with similar design standards [Kihlberg and Tharp 1968].

4. Experience shows that the accident rate on uncontrolled access highways generally is twice that on controlled access highways [AASHO 1965].

5. Medians reduce the number of head-on collisions. Median barriers or guardrails, however, may increase the accident rate while lowering the severity of the accidents [Kihlberg and Tharp 1968, Moskowitz and Shaefer 1960].

6. The accident rate on urban roads is about twice that on rural roads. The accident rate on suburban roads is about 1-1/2 times that on rural roads [Winfrey 1969].

7. Accident rates are thought to be proportional to both the gradient and the degree of curvature, i.e., steeper grades and sharper curves have higher accident rates [Kihlberg and Tharp 1968, Winfrey 1969, Bitzl 1957]. See Table 7.5 for approximate relationship.

8. The assumption is made that the wet-weather accident rate (i.e., the number of wet-surface accidents per 100 million vehicle miles (161 million vehicle kilometers) travelled under all conditions) is directly proportional to the amount of rainfall in a given region. The rainfall map in Figure 7.3 was taken from Bock et al. [1972] and modified by the authors to include suggested correlation factors for various regions.

Once the number of accidents per mile (1.61 km) of highway has been estimated for a given situation, the cost of these accidents must be approximated. Assigning costs to accidents has been the subject of numerous studies. These studies have resulted in widely different costs due to disagreements as to which elements of accident costs should be considered. For the

Table 7.5. Accident Rates for Horizontal Curves and Vertical Grades [Winfrey 1969]

Degree of Curve or Percent Grade	Ratio of Accident Rate to Rate on Level Tangents: Horizontal Curves	Ratio of Accident Rate to Rate on Level Tangents: Vertical Grades
0	1.0	1.0
1	1.3	1.1
2	1.6	1.5
3	2.0	2.0
4	2.4	2.7
5	2.9	3.7
6	3.6	4.9
7	4.3	6.2
8	5.2	7.6
9	6.3	9.2
10	7.6	10.8
11	9.1	12.4
12	10.8	14.1
13	12.5	
14	14.4	
15	16.3	

Note: For a combined horizontal curve and vertical grade, use the sum of the two ratios for their respective values. For example, for a 4 degree curve and a 3 percent grade the ratio of accidents to the level tangent accidents would be 2.4 plus 2.0, or 4.4.

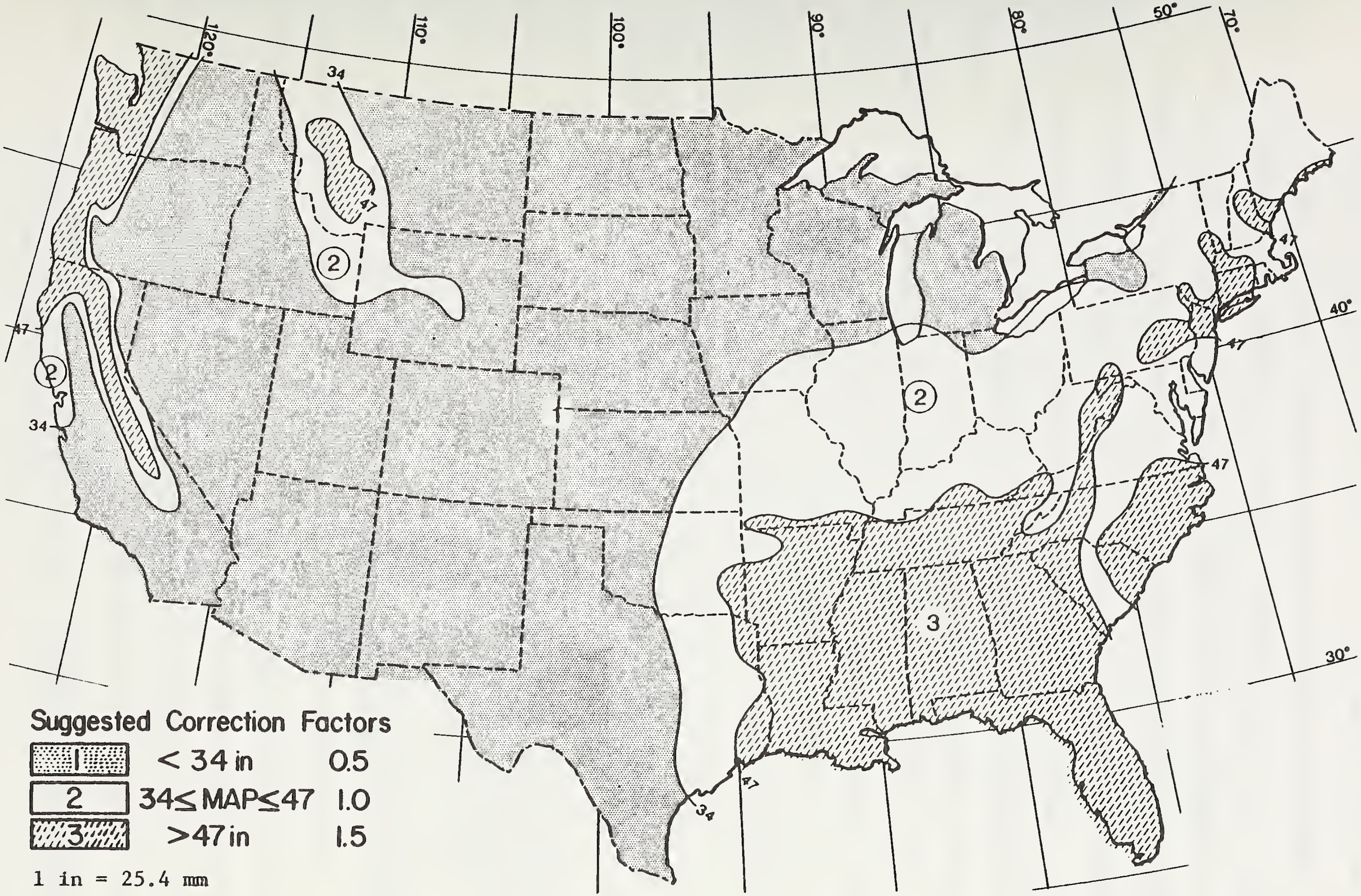


Figure 7.3. Correction Factors for Regional Mean Annual Precipitation (MAP)

economic analysis at hand, it is not vital that the issues be resolved because a sensitivity analysis employing a range of accident costs can be conducted. As a general guide, however, the AASHO average cost of accidents is frequently cited. AASHO considers the average direct cost of a reported urban traffic accident to be $700 [AASHO 1973]. If $1,000 were used as the average direct cost of a reported suburban accident, and $2,000 for a reported rural accident, then these figures would be consistent with data provided by the National Safety Council and various other studies [Winfrey 1969, Campbell and Titus 1971].

Thus, if one wanted to estimate the yearly cost of wet-surface accidents on one mile of urban, two-lane, uncontrolled access highway in Florida having a speed limit of 45 mph (72 km/h), 7000 ADT, a 3° horizontal curve but no vertical grade, no median barrier, and SN_{40} = 50, he would first enter Figure 7.2 with SN_{40} = 50 and find that the wet surface accident rate is 30 accidents per 100 million vehicle miles (161 million vehicle kilometers) under the Rizenbergs conditions. The following correction factors would be applied next:

Correction factor for urban condition	2
Correction factor for noncontrolled access	2
Correction factor for horizontal curve	2
Correction factor for rainfall	1.5
Total correction factor (2x2x2x1.5)	12

(Note that no correction factor is needed for speed limit, median barrier, and number of lanes.)

Therefore, the corrected wet-surface accident rate is 30x12 or 360 wet-surface accidents per 100 million vehicle miles (161 million vehicle kilometers). To convert this rate into a number of wet-surface accidents per year, one must multiply the rate by the total number of vehicle miles (or vehicle kilometers) travelled over the highway in one year:

$$\frac{\text{360 wet-surface accidents}}{\text{100 million vehicle miles*}} \times \text{7000 ADT} \times \text{365 days} = \text{9,2 wet-surface accidents per mile* per year}$$

* 1 mi. = 1.61 km

This rate of accidents is in terms of reported wet-surface accidents. Although the proportion of unreported accidents is high, their cost is low--thus, unreported accidents are disregarded here. Since the average cost of a reported urban accident is $700, the estimated yearly cost of wet-surface accidents on this particular highway is about $6,400.

Of course, it is possible to develop much more accurate cost estimates for examples such as the one just presented. The underlying assumptions and the selection of correction factors can be made more detailed; also, more variables can be included if data showing accident correlation are available ble. However, the method presented above has been kept as simple as possible in order to reduce to a minimum the number of representative situations which can be encountered.

One other comment should be made about the above example. The example is somewhat misleading because it implies that total wet-accident costs must be computed in the economic analysis. However, that is not the case. The example was given simply in order to provide a better understanding of the benefit associated with a reduction in accidents. This benefit is the saving in costs as a result of increased skid resistance. It can be seen in Figure 7.2 that the accident rate decreases by about 2.0 for each unit increase in skid number up to $SN_{40} = 45$; then it decreases by about 0.4 for an SN_{40} greater than 45. This applies to either of the ADT levels shown in Figure 7.2. It is the only information which is needed from Figure 7.2 for the economic analysis. Thus, in the above example, the saving in accident costs is simply the increase in SN_{40} multiplied first by 0.4, then by all the necessary factors.

7.3.2 Noise

Noise is defined as unwanted sound. The physical measure of sound is in terms of decibels measured using the A-weighting network of a standard sound level meter. This measure is commonly referred to as the A-weighted sound level, dBA.

The problem of quantifying noise pollution is largely a subjective one. In this analysis, it will be assumed that noise can exist only where people are present. With no people to judge a sound as unwanted, there can be no noise. Therefore, a highway with a high A-weighted sound level is not considered noisy in this analysis unless there are peopled exposed to the sound.

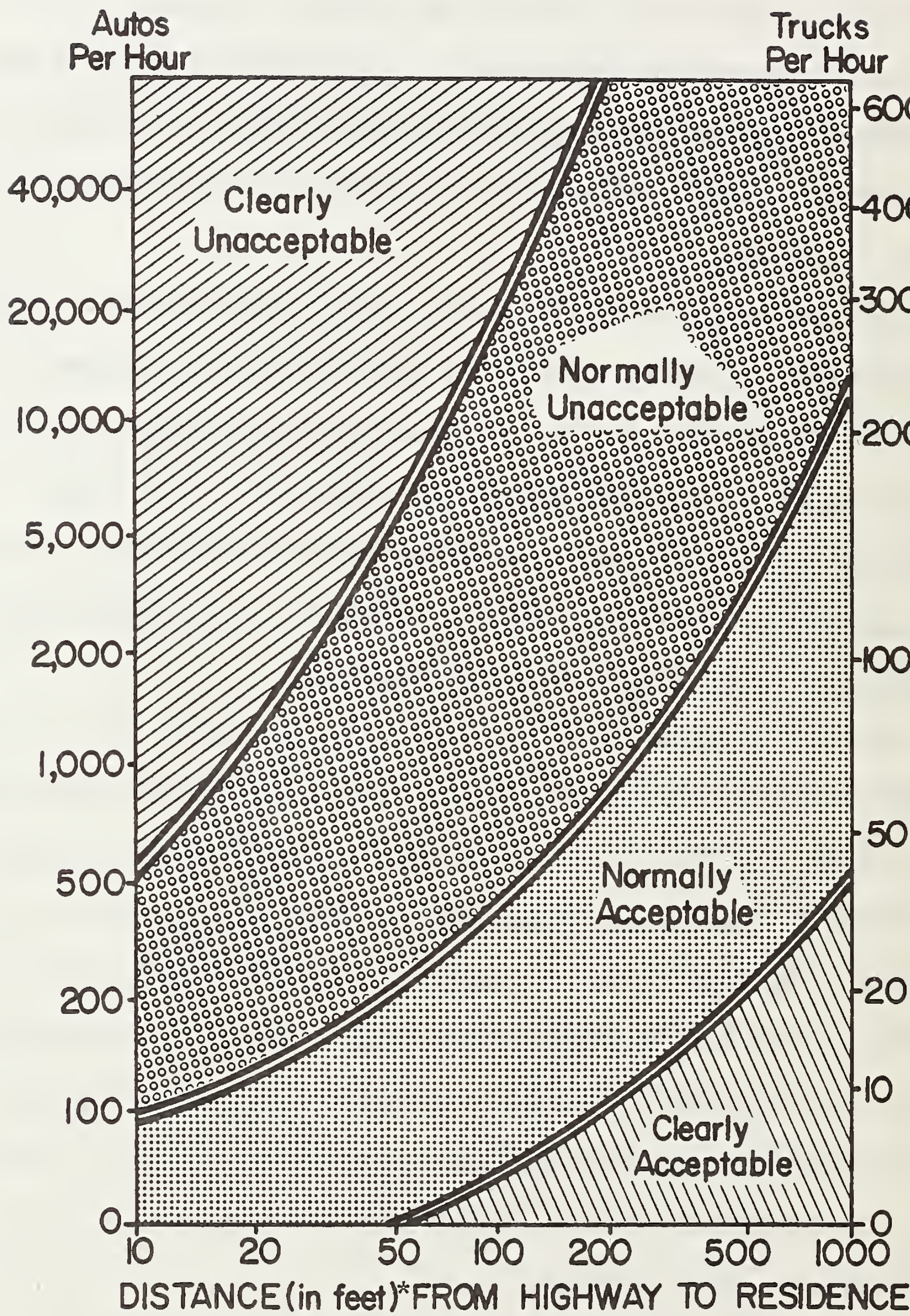
It has been shown that the factors which affect the sound level of a highway include vehicle speed, number of lanes, density of traffic, proportion of trucks to cars, depression or elevation of roadway, steepness of grade, road texture, vegetation, noise barriers, and distance to measurement site. When these factors are known, sound-level estimates of the highway can be made. The impact of the objectionable sound, or noise, can then be estimated in view of the surrounding land use. Although it is possible to estimate highway noise in a given situation, it is not necessary that this be done for the purpose of this analysis. As with accidents, what is desired for the economic analysis is the benefit, or in this case, the difference in noise between two alternative resurfacing systems.

Table 4.3 will be used as a guide to estimate the difference in traffic noise as a result of varying the road texture in a given situation. Although the table is somewhat crude, it is still perhaps the best available in light of the lack of data relating sound level to specific resurfacing systems. Using Table 4.3, one must assign to the resurfacing system being evaluated an adjustment ranging from -5 to +5dB. The exact value of this adjustment is subjective and is based upon one's knowledge of the resurfacing system being evaluated. The section on noise should provide useful information for making a suitable estimate of the actual adjustment.

Once the relative sound level has been determined for a given situation, two questions logically follow: 1) Is an increase or decrease in sound to be considered as noise in the situation being examined and 2) If it is noise, are the traffic conditions such that changes in surface texture will have an effect upon the noise level? For the purposes of this study, several assumptions and simplifications will be made to provide a workable, easy-to-use set of guidelines for answering these two questions.

Figure 7.4, taken from Schultz and McMahan [1971], will provide some insight into the first question. In Figure 7.4, it can be seen that a housing site located 100 feet (30 meters) from the highway would be rated as clearly unacceptable from a noise standpoint for peak hourly traffic flows of 20,000 automobiles or 300 trucks. In general, Figure 7.4 indicates that the hourly traffic must be fairly large for the sound level to be considered unacceptable unless the residences are located extremely close to the highway. Even when the distance from the highway to the residence is 10 feet (3 meters), the hourly traffic must still be greater than 100 automobiles in order for the sound level to be unacceptable.

In view of Figure 7.4, the assumption will be made that noise is of no concern in the case of rural roads. This assumption is fairly realistic



* 1 ft = 0.3 m

Figure 7.4. Highway Noise Impact on Residences
[Bolt, Beranek, and Newman, Inc.]

considering the fact that about two-thirds of all the rural roads in the United States carry fewer than 400 vehicles per day [Oglesby and Altenhofen 1969]. Furthermore, because of the lower population densities, the presence of residences located extremely close to the roadway would not appear to be significant. In the case of urban and suburban roads, however, it will be assumed that noise is a problem when the ADT is 40,000 or higher or when the ADT is at least 2,000 and residences are located within 10 feet (3 meters) of the highway.

Now for the second question: when highway noise is a problem, under what traffic conditions will the noise level change with changes in surface texture? To answer this question, one should know the degree to which the tire-pavement noise component contributes to the overall A-weighted sound level. If the tire-pavement noise component is not a major source of noise in a given situation, then changes in surface texture will not appreciably affect the noise level. It was stated in Chapter 4 that the tire-pavement noise component is a factor only for those cases of high volume traffic containing low percentages of trucks. In consideration of this fact, the additional assumption is made that the effect of pavement texture on noise is negligible where the traffic stream contains a truck percentage greater than 5 percent.

The assumptions made thus far in this noise model have been somewhat arbitrary. It may be argued that the other factors previously listed as influencing the sound level should also be considered when predicting locations having unacceptable noise levels. The authors, however, believe that there is no need to employ a detailed model to predict highway noise because there are difficulties that arise elsewhere in the noise model, which will cancel any attempt at achieving accuracy. For instance, the problem of converting noise into monetary terms has no solution which is fully acceptable--yet this problem must be addressed in the noise model. Therefore, while it is possible to predict accurately the sound level given in specific situations, the noise model used here will employ only the ADT, the distance from the highway to the residence, and the percentage of trucks. Thus, the only factors which must be known in addition to those which are already required for the accident model are the distance from the highway to the residence and the percentage of trucks.

The problem of converting noise into dollar terms must now be discussed. No technique is presently available for placing market prices on the benefits received through a reduction in the A-weighted noise level within a community [Piersol and Winfrey 1974]. If it is assumed that action will always be taken to prevent a noise problem or to correct it where it does exist, then the cost of reducing an unacceptable noise level to a point at which it is acceptable may be thought of as the cost of noise. The assumption that preventive or corrective action will be taken is probably realistic in view of the National Environmental Policy Act of 1969, which has spurred highway designers and transportation departments to become noise conscious.

Several strategies have become acceptable for reducing the noise generated by a highway. These strategies are normally grouped in three categories: 1) reduction of noise from motor vehicles; 2) adequate land-use zoning adjacent to highways; and 3) proper highway design and location to protect persons from excessive noise. Where necessary, strategies within all three of these categories have been simultaneously employed.

For existing highways, after strategies within the first two categories have been employed, the only possible remaining solutions aimed directly at highway design are: 1) resurfacing, 2) erection of roadside barriers, 3) depression of the roadway, and 4) elevation of the roadway on a fill or on a structure. The latter two alternatives are not normally recommended except in extreme cases because they are not very cost-effective [Piersol and Winfrey 1974]. Therefore, when the highway is resurfaced, the savings in the cost of noise can be assumed to be the savings resulting from not having to erect a roadside barrier of equivalent effectiveness. It has been estimated that each roadside barrier costs $4,000 per mile (1.61 km) per year for each dBA of noise reduction desired [Piersol and Winfrey 1974]. Thus, if a 2 dBA reduction in noise is desired, the roadside barrier will cost $8,000 per mile (1.61 km) per year. It can therefore be said that the noise benefit associated with a resurfacing system that reduces noise by 2 dBA is $8,000 per mile (1.61 km) per year.

7.3.3 Maintenance

Maintenance has been defined by AASHO as consisting of two subdivisions: physical maintenance and traffic services [AASHO 1958-60]. Physical maintenance is "the act of preserving and keeping a highway, including all its elements, in as nearly as practicable its original, as-constructed condition or its subsequently improved condition." Traffic services, on the other hand, are defined as "the operation of a highway facility and services incidental thereto, to provide safe, convenient and economical highway transportation." In computing the benefit of decreased maintenance for this report, a more restrictive interpretation has been adopted. Only physical maintenance is considered to any extent here, and this maintenance deals only with work that results from changes or variations in resurfacing systems. Also excluded from maintenance in this report are activities ordinarily identified as betterment, construction, and reconstruction. Traffic services are normally not influenced by the type of pavement resurfacing and therefore need not be covered in any detail here.

There exists little useful data in the literature for predicting the maintenance expenditures associated with various resurfacing systems. Two major reasons can be given: 1) Maintenance expenditures vary significantly from one highway agency to another because of differences in policies, standards, and effort expended; and 2) No single property of the resurfacing system has yet been found which can satisfactorily be correlated to the amount of required maintenance for all possible resurfacing systems.

The procedure that has been chosen in this report for estimating maintenance benefits is similar to that discussed earlier for estimating the initial cost of a resurfacing system. In other words, the estimator must have some knowledge of the resurfacing system to enable him to define the elements (in this case, the maintenance work elements) that differ for the two resurfacing systems being compared. For example, when comparing an asphalt-rubber seal coat with a conventional asphaltic seal coat, the maintenance expenses for the two systems will be similar except that the asphalt-rubber seal coat, being less brittle and more waterproof, will probably require less sealing of reflective cracks and fewer repairs over areas with an expensive subgrade program. After appropriate maintenance work elements have been defined, an estimate must be made of the magnitude of the reduction in maintenance work. This estimate will of course be dependent on factors such as ADT, geographic location, urban or rural setting, etc. The maintenance performance standards of a highway agency can be consulted to arrive at the manhour, equipment, and material requirements; these can then be converted into dollars.

Table 7.6 can be used as a checklist to help the estimator determine which maintenance work elements need to be investigated. Representative costs and production rates are also provided in this table.

Table 7.6. Maintenance Work Elements and Their Representative Costs [Adapted from Butler 1974]

Maintenance Work Element	Costs: Labor and Equipment[a]	Costs: Material[b]	Production Rate
Crack cleaning and sealing (bituminous material)	$30-50/hr	$.08-.15/lf	500-800 lf/hr
Joint cleaning and sealing	$26-40/hr	$.06-.10/lf	200-300 lf/hr
Surface patching (hand patch with bituminous cold mix)	$55-70/hr	$.30-.40/sy	50-80 sy/hr
Surface patching (bituminous hot mix - mechanized)	$50-60/hr	$.40-.50/sy	175-225 sy/hr
Partial depth patching (bituminous hot mix)	$30-35/hr	$.80-1.00/sy	11-20 sy/hr
Partial depth patching (Portland cement concrete)	$30-35/hr	$.20-.25/sf	5-10 sf/hr
Full depth patching (Portland cement concrete)	$45-50/hr	$6.00-9.00/sy	10-20 sy/hr
Surface treatment (liquid bituminous and aggregate - manual)	$45-50/hr	$.30-.50/sy	70-80 sy/hr
Surface treatment (liquid bituminous and aggregate - mechanized)	$55-65/hr	$.30-.50/sy	500-650 sy/hr
Repair of blowups	$45-55/hr	$70-90/site	.3-.6 sites/hr
Cleaning of grooves	$15-20/hr	--	200-300 sy/hr
Correcting raveling and stripping (protective coating or surface enrichment - mechanized)	$10-15/hr	$.25-.40/sy	1500-2000 sy/hr
Corrective bleeding (hot sand or screenings)	$10-15/hr	$.10-.25/sy	1500-2000 sy/hr
Corrective scaling (slurry seal - applied by squeegee)	$15-20/hr	$.30-.50/sy	100-150 sy/hr

Other maintenance work elements that may also be considered:

Correcting popouts (partial depth patching)
Cleaning (consider especially intersections and bridges)
Snow removal (consider amount of de-icing agent required)
Traffic striping (consider adherence and type of striping required)

[a] Labor and equipment costs do not include costs of traffic control and office overhead

[b] lf = 0.3m; 1 sy = 0.84 sq. m

7.3.4 Vehicle Operating Costs

Vehicle operating costs include the cost of fuel, tires, oil, and maintenance and repair. Several studies have shown that surface roughness affects vehicle operating costs [Claffey 1960, Claffey 1965, Claffey 1971, and Sawhill and Firey 1960]. However, roughness is of a magnitude beyond that of surface texture, and it is surface texture that should be used as the variable to distinguish between the resurfacing systems in this economic analysis. Thus, while these studies provide some insight into the possible relationship between surface texture and vehicle operating costs, they cannot be used to formulate a quantitative model for vehicle operating costs.

Of the four vehicle operating costs mentioned above, tire wear appears to be the most important in terms of sensitivity to variation in surface texture, and it is the only one that will be discussed to any extent here. First, however, a justification for the elimination of the other operating costs from the economical analysis model will be presented.

The costs associated with fuel consumption have been shown to increase appreciably on rough gravel or loose stone surfaces as extra energy is needed either to force the wheels over the stones or to push the stones aside. On extremely rough paved roads, fuel consumption is increased primarily as a result of speed changes to avoid potholes and other broken surfaces. In this study, it will be assumed that the resurfacing systems being evaluated will not be allowed to deteriorate to the point where changes in vehicle speed are required; thus, fuel consumption differences will not be considered here.

Oil consumption and maintenance are primarily dependent on the dust-producing characteristics of road surfaces--the more dusty the surface, the greater the frequency of engine oil changes along with an increase in the rate of wear of cylinder walls, piston rings, bearing surfaces, etc. Of course, badly deteriorated roads will increase the maintenance and repairs on vehicle suspension systems, but it is not believed that minor variations in surface texture will significantly affect the cost of vehicle maintenance. Thus, neither oil consumption nor maintenance will be considered.

While the authors believe the costs of tire wear may be a significant factor in the economic analysis, there exists little in the literature to enable the quantification of tire wear costs as related to surface texture. A model will nonetheless be presented from the available data, but it is suggested that the model be incorporated into the economic analysis at the option of the user.

The finding [Ford Motor Co. 1973, Bond et al. 1974] that tire wear (or more correctly, tire abrasion) is mainly a function of microtexture is significant. Because the abrasiveness of a road surface increases with an increasing number of microtexture asperities in contact with the tread rubber, the conclusion that surfaces having good friction properties (i.e., high skid numbers) will cause greater tire wear appears to logically follow.

A model based solely on the relationship between tire wear and skid number (e.g., SN_{40}) is probably an oversimplification, however. In open-graded courses, for example, drainage facilitated by the macrotexture enables the microtexture asperities to penetrate through the water film when the surface is wet. Thus, an open-graded surface having little microtexture may, under certain wet conditions, wear tires more than a dense-graded surface having a higher level of microtexture; under dry conditions, the reverse may occur, even though the SN_{40} for the two surfaces is the same. Thus, a realistic model for tire wear should ideally be based not on SN_{40} but on the skid number (both wet and dry) at a speed more representative of traffic conditions. Further, it would be desirable to have a knowledge of such variables as the frequency and intensity of rainfall, the ambient or pavement temperature, the traffic maneuvers (e.g., cornering, stopping), etc., in order to obtain more accurate predictions of tire wear.

The increased complexity that quickly results, together with the lack of research data to establish the necessary relationships, has caused the authors to develop a model based on SN_{40}, but because of uncertainties associated with the limited data base, the model should be regarded as tentative until more research has been conducted. As mentioned earlier, this model is to be considered an optional one; it is not included in the cost-benefit example at the end of this chapter.

Table 7.7 is used to establish a tire wear model. It can be seen from Table 7.7 that speed is not a major influence on the difference in tire wear costs between the concrete and the asphaltic surfaces; i.e., the cost of tire wear is roughly a constant 0.15 cents per mile (1.61 km) greater on the asphaltic surface than on the concrete surface. It is important to note that the concrete surface of Table 7.7 had a coefficient of friction of 0.36 (wet), and the asphaltic surface had a coefficient of friction of 0.48 (wet).* Because of the lack of any better data, a straight-line relationship is assumed to exist between tire wear costs and the coefficient of friction (and therefore the SN). It is also assumed that the difference in tire wear costs for the two paved surfaces of Table 7.7 is 0.25 rather than 0.15 cents per mile (1.61 km) to reflect recent price increases. Simple arithmetic thus yields an increase in tire wear costs of about 0.02 cents per mile (1.61 km) for every unit increase in SN. For example, an increase in SN of 5 for a 7000 ADT road results in the following yearly increase (or a negative benefit) in tire wear costs to the user:

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* The tire wear tests were conducted under dry conditions, but dry friction coefficients were not reported.

$$.02\ \left(\frac{\text{cents/mile}}{\text{SN}}\right) \times 5\ (\text{SN}) \times 7000\ (\text{ADT}) \times 365\ (\text{days})$$

$$= \$2555 \text{ per mile } (1.61 \text{ km}) \text{ per year.}$$

This is significantly greater than the positive benefit of reduced accidents as a result of increasing SN by 5 on a highway having the conditions given in Section 7.3.1 of this report (about $430 per mile (1.61 km) per year).

Again, it must be stressed that certain simplifying, although perhaps not accurate, assumptions had to be made because of the lack of research data dealing with the influence of SN (or any other characteristic of specific resurfacing systems) on vehicle operating costs. As more data become available, more accurate and detailed cost estimates will be possible.

Table 7.7. Automobile Tire Cost as Affected by Speed and Type of Surface--Straight Road and Free-Flowing Traffic [Claffey 1971][(1)]

Uniform Speed (mph)	Cost of Four Tires (cents/mile or 1.61 km): High Type Concrete u = .36	High Type Asphalt u = .48	Dry Well-Packed Gravel
20	0.09	0.27	1.03
30	0.19	0.36	1.05
40	0.29	0.43	1.07
50	0.32	0.45	1.10
60	0.31	0.46	----
70	0.30	0.44	----
80	0.27	0.43	----

(1) All tire wear costs are for dry surfaces.

7.3.5 Other Benefits

Thus far in this report, the benefits associated with decreases in accidents, noise, vehicle operating costs, and maintenance have been discussed. A method for converting these benefits into dollar savings has been presented for each of the benefits. The objective is to add for each resurfacing system being evaluated all of these monetarily quantifiable benefits in order to obtain a sum, which, when divided by the cost of the resurfacing system, will yield a benefit-cost ratio. This benefit-cost ratio, however, cannot be used by itself to determine the relative desirability of a particular resurfacing system. For one thing, the ratio as described above does not provide complete information about benefits. It reflects only monetarily quantifiable benefits. Benefits that cannot be valued in dollar terms do not appear in the benefit-cost ratio. For this reason, many investigators believe the benefit-cost ratio should be used only as an aid to the decision maker and not to make the decision for him.

What other benefits, then, should enter the decision process in this study, and how should these be evaluated? The answers to these questions are dependent upon the particular resurfacing system being analyzed. The more that is known about the advantages or disadvantages of a resurfacing system, the easier it will be to identify additional benefits for consideration. These additional benefits may be of the type discussed thus far (i.e., monetarily quantifiable benefits), in which case they would be included in the benefit-cost ratio; or they may not be monetarily quantifiable in which case they may each be assigned a benefit-cost ratio and considered formally in an analysis (provided they are quantifiable), or they may be lumped together and considered subjectively (if they are not quantifiable).

Because of the obvious difficulty of identifying the benefits to be incorporated in a model that evaluates innovative resurfacing systems and resurfacing systems which have yet to be developed, the authors can only discuss those benefits which are likely to be a factor in the analysis of many of these resurfacing systems. Of course, benefits other than those mentioned below may also be important in certain cases and should, by all means, be given adequate consideration.

7.3.5.1 Maintenance-Caused Motorist Delays

The benefit associated with maintenance-caused motorist delays in this study refers primarily to the increase (or decrease) in the amount of time spent by motorists on the road as a result of interference by maintenance crews involved in the repair of damaged highway surfaces. The delays frequently take the form of slowing down the motorist as he passes the work site, sometimes stopping him momentarily or rerouting him on a less desirable road. Because travel time has a value, this benefit can be monetarily quantified. Considerable research, in fact, has been done to estimate the value of time for passenger cars and commercial vehicles [Adkins, Ward and McFarland 1967, Curry and Anderson 1972, Haney, 1963, Thomas 1967].

A major problem encountered in this study, however, is that motorist delays due to roadway occupancy by maintenance forces cannot be quantified unless the specific conditions are known. For example, the road being repaired and the nature of the work may be such that traffic can remain open on several existing lanes; it may also be the case that traffic must be detoured to another road which, depending upon the length of the detour, can itself be a cause of wide variations in delay time; or it may be best to construct a temporary lane or a temporary road parallel to the highway. What is done affects not only the magnitude of delays in time, but also the cost of maintenance. The more maintenance work that must be performed, the more often traffic must be rerouted, and therefore, the more it will cost to put up detour signs, etc. (This element of maintenance cost, i.e., the cost of rerouting traffic, was in fact purposely left out of the maintenance benefit model because of the large possible fluctuations in costs.) Still another consequence which must be considered is the possibility of an increase in accidents when motorists are forced to travel over unfamiliar and often inadequate detours. Changes in speed, as for example, slowdowns near maintenance roadway occupancy sites, may also be responsible for an increased accident rate. The point of this discussion is that maintenance-caused motorist delays and their accompanying costs cannot be estimated for the general representative situations that are examined in this economic analysis. Maintenance-caused motorist delays therefore are not quantifiable in the framework of this study and cannot be included in the benefit-cost ratio. Nevertheless, when decreased maintenance is a significant benefit, the resulting decrease in motorist delays may also be significant and should be given subjective consideration.

7.3.5.2 Conservation of Natural Resources

A skid and wear resistant resurfacing system which not only has a relatively high benefit-cost ratio but also optimizes the use of natural resources is certainly a desirable system and one that should be striven for. The benefit stemming from the conservation of scarce natural resources through the use of a particular resurfacing system is difficult to quantify. Any attempt at quantification should ideally involve a systems approach utilizing a knowledge of: 1) the amount, type, and location of our natural resource reserves, 2) the expected proportion of each type of resurfacing system to be employed, and 3) the natural resource requirements for each type of resurfacing system. Implied in such a systems approach is the presumption that the optimum use of natural resources is not necessarily obtained through the use of one given type of resurfacing system but through the combined use of several types of resurfacing systems. At any rate, the complexity involved in such a method of quantification is not only beyond the scope of this research, but it is not extremely vital for the economic analysis at hand. It will be recalled that the objective of the economic analysis is to identify some optimal resurfacing systems by first eliminating from the list of possible wear and skid resistant systems those with a low benefit-cost ratio and then further eliminating those whose monetarily unquantifiable benefits appear to be decidedly negative. When the expected use of a resurfacing system (i.e., the purpose and extent of its use) is known, its impact upon the conservation of natural resources can be predicted qualitatively with sufficient accuracy to meet the needs of this economic analysis.

It is important that the influence of the resurfacing system on the use of natural resources be investigated for all possible aspects of the resurfacing system. Consideration should be given to the materials used, production processes and treatments, transportation, method of laydown, texture application, and maintenance requirements, for these are the primary sources of natural resource consumption. Some general information will be presented in order to better illustrate the considerations which must be made in this section.

Much has been written on optimizing the use of natural resources in highway construction [Transportation Research Board 1976]. It is clear that while our aggregate reserves are literally inexhaustible, their geographic distribution often does not match requirements, leaving many areas in the U.S. with a shortage of quality aggregate. Hauling aggregate from one region to another can consume large quantities of energy, and aggregate that is normally inexpensive can become quite costly. Thus, the need for supplementary aggregates varies with the region of the U.S. The use of manufactured aggregates in regions where the sources of aggregate have been depleted is, of course, a possible alternative. The purpose of such an alternative aggregate would not be to meet the needs of the U.S. in general but those of a specific region. It is therefore not necessary that large quantities of one type of material be expended, if the purpose of the alternative is to meet localized needs.

The energy requirements of various resurfacing systems vary considerably. Some aggregates do not require heating or drying prior to incorporation in the mix. Other processing requirements may also vary widely. Binding agents are especially energy-intensive. A rapidly increasing strategy that conserves the use of energy is to use a binder that combines two or more materials. Lime and lime-flyash or other pozzolanic systems combined with Portland cement are an example. Another example is the use of sulfur-asphalt combinations. It is also possible that alternative binders that can be used singly and require smaller amounts of energy will be developed. Sulfur, for example, is such a new binder which may find substantial use. Cutbacks, on the other hand, evaporate tremendous amounts of diluents, naptha, and kerosene to the atmosphere and are slowly being replaced where possible by emulsions, hot asphalt cement, etc.

Many engineers believe that the conservation of natural resources can best be accomplished by utilizing wastes in construction. Indeed, waste materials, by-products, and recycled products have had successful applications and will probably be used more extensively in the future. The benefits associated with using wastes can be significant, since these wastes would otherwise be disposed of, a burden which costs both money and energy. In the economic analysis at hand, the use of waste products in a resurfacing system is a benefit which is partially reflected in the cost of the system (and therefore in the benefit-cost ratio), but it must also be given credit for its role in the conservation of natural resources.

7.3.5.3 <u>Miscellaneous Benefits</u>

There are a number of other possible benefits which should be given consideration when the situation arises. These are benefits which are related to some secondary property of the resurfacing system. These benefits should probably not be investigated unless the characteristic which measures the property is at one of the extremes. For example, the reflection properties of most conventional aggregates are such that the reflectivity is small and fairly uniform from one aggregate to another. However, if a highly reflective manufactured aggregate is developed, the resulting benefits can be significant: the road surface is better contrasted with its surroundings, adding to vehicle and pedestrian safety; also, the amount of power needed to illuminate city streets and intersections can be reduced, resulting in savings in cost. Although the benefit of reflection is thus related to accidents and to maintenance (traffic service) costs, it was not taken into account in those benefit models. Therefore, the benefit of reflection, and other such miscellaneous benefits that may be applicable should enter into the decision process at this time. Table 7.8 lists several miscellaneous benefits that can be of significance. The list is by no means complete, however, and it is suggested that any other extreme in a property characteristic also be given consideration.

7.4 Benefit-Cost Tabulations

7.4.1 Conventional Resurfacing Systems

A set of examples will now be worked out to show how the cost and benefit models just described can be put to use in an economic evaluation of resurfacing systems. This evaluation will be conducted for several field-proven, conventional resurfacing systems. The purpose of this evaluation is twofold: first, to test, through the use of familiar resurfacing systems, whether the overall benefit-cost model provides realistic results; and second, to establish a basis for comparison which may be used to measure the innovative resurfacing systems that eventually will be subjected to economic analysis.

The conventional resurfacing systems that have been chosen for the evaluation can be classified as 1) a continuously reinforced concrete overlay, 2) a dense-graded bituminous concrete overlay, 3) an open-graded bituminous concrete overlay, 4) a bituminous concrete overlay with sprinkle treatment, and 5) an asphalt-rubber seal coat. Within each class a differentiation can then be made according to type of aggregate, i.e., for each class, aggregate type can be varied. The aggregates that were chosen for inclusion in this evaluation are: 1) crushed stone, 2) metallurgical slag, 3) expanded slag, and 4) calcined bauxite. Thus, the evaluation consists of 20 resurfacing systems--five classes, each having four types of aggregates.

Table 7.8. Miscellaneous Benefits

Property	Associated Benefits (Positive and Negative)
1. Reflection	Highly reflective surface increases contrast and delineation (safety); also reduces illumination requirements ($).
2. Glare	Surface that has glare causes a decrease in night vision (safety).
3. Thermal Conductivity	Low thermal conductivity decreases depth of frost penetration; high thermal conductivity decreases warping and cracking (maintenance).
4. Electric Conductivity	Low electric conductivity does not allow static electricity from motor vehicles to dissipate through the pavement (safety).
5. Thermal Expansion	High coefficient of thermal expansion increases buckling and blowups; if coefficients of thermal expansion of individual components are not similar, an increase in differential expansion or contraction cracks results (maintenance).
6. Density	Low density permits the resurfacing system to be employed on bridges with dead load limitations; also lower transportation costs and energy savings are possible ($).
7. Degradation	High resistance to degradation decreases the damage done by loose particles flung into traffic (safety).
8. Chemical Reactivity	High chemical reactivity can seriously affect the durability of a resurfacing system; high resistance to salt and oil dripping deterioration permits the resurfacing system to be employed where salt and oil drippings are anticipated (maintenance).
9. Appearance	Brilliant colors can be distracting to the driver; these colors may however be desired in caution areas such as exit ramps (safety).
10. Drainage	Surfaces with good drainage capabilities, for example, open-graded courses, allow for high speed skid resistance and less splash and spray (safety).

Tables 7.9a through 7.9f provide sample benefit-cost evaluations of the alternative resurfacing systems in question. Each of these tables is associated with a specific set of conditions. Table 7.9a, for instance, is for a situation involving a two-lane rural highway carrying an ADT (two directional) of 5000 vehicles per day, having controlled access to the highway, no curves or grades, no noise problem, and being situated in a zone of medium rainfall (i.e., zone 2 from Figure 7.3). These specific conditions must be defined in order to use the cost and benefit models that have been discussed. The reason for having established extremely simplistic benefit models should also be apparent now. Had the models been made more detailed by including more parameters, greater accuracy would, of course, have been possible, but the economic evaluations would have had to be defined for a large, unmanageable set of specific conditions.

The benefit-cost tabulations will now be explained using Table 7.9b as an illustration. First, each alternative resurfacing system must be fully identified. An identification such as "a 5 in. (127mm) thick continuously reinforced concrete overlay utilizing expanded slag coarse aggregate (aggregate type 3)" is needed in order to estimate both the costs and the skid-and wear-resistance characteristics. Recommended also in this identification is the type of binding agent, aggregate proportions, and other differentiating features of the resurfacing system as shown in Figure 7.1. In the evaluations of Table 7.9 it is assumed that the binding agent is either AC-20 asphalt cement, Type I or Type II Portland cement (611 lb/yd^3, 365 kg/m^3), or asphalt-rubber in the case of the asphalt-rubber seal coat. It is also assumed that the amount of coarse aggregate used in the mix is 2000 lb/yd^3 (1200 kg/m^3) except in the case of expanded slag where the

Table 7.9a. Benefit-Cost Tabulation for Several Conventional Resurfacing Systems

Case 1 a) 2-lane highway, each lane 12 ft wide
b) rural area
c) controlled access
d) no curves, no grades
e) medium rainfall (zone 2)
f) ADT = 5000
g) no noise problem

	CRG Overlay (5" thick)[2]				Dense-Graded Bituminous Concrete Overlay (1-1/2" Thick)				Open-Graded Bituminous Concrete Overlay (3/4" Thick)			
Aggregate Type[1]	1	2	3	4	1	2	3	4	1	2	3	4
Initial Cost ($/S.Y.)[2]	7.00	6.96	8.28	12.20	1.38	1.39	1.48	2.95	0.80	0.79	0.85	1.60
Initial Cost ($/mi.)[2]	98,560	97,999	116,582	171,776	19,360	19,571	20,838	41,536	11,264	11,123	11,968	22,528
Estimated Life (years)	12	12	12	12	10	10	10	10	8	8	8	8
Yearly Cost per Mile	14,469	14,386	17,114	25,217	3150	3184	3390	6758	2111	2084	2243	4222
Relative Yearly Cost	12,385	12,302	15,030	23,133	1066	1100	1306	4674	27	0	159	2138
Skid No. Range	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65
Average Skid No.	38	52	52	58	38	52	52	58	38	52	52	58
Relative Yearly Accident Benefits($)	-613	0	0	+88	-613	0	0	+88	-613	0	0	+88
Relative Yearly Maintenance Benefits ($)	+210	+210	+210	+210	0	0	0	0	0	0	0	0
Relative Yearly Noise Benefits($)	-	-	-	-	-	-	-	-	-	-	-	-
Total Relative Yearly Benefits ($)	-403	+210	+210	+298	-613	0	0	+88	-613	0	0	+88
Relative Benefit-Cost Ratio	neg	.02	.01	.01	neg	0	0	.02	neg	std	0	.04
Relative Yearly Benefits, Minus Costs	-12,788	-12,092	-14,820	-22,835	-1679	-1100	-1306	-4586	-640	0	-159	-2050
Recommended Systems										✓	✓	

	Bituminous Concrete Overlay (1-1/2" Thick) with Sprinkle Treatment				Asphalt-Rubber Seal Coat			
Aggregate Type[1]	1	2	3	4	1	2	3	4
Initial Cost ($/S.Y.)[2]	2.85	2.85	2.85	2.95	0.95	0.95	1.00	1.50
Initial Cost ($/mi.)[2]	40,128	40,128	40,128	41,536	13,376	13,376	14,080	21,120
Estimated Life (years)	10	10	10	10	6	6	6	6
Yearly Cost per Mile	6529	6529	6529	6758	3071	3071	3233	4849
Relative Yearly Cost	4445	4445	4445	4674	987	987	1149	2765
Skid No. Range	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65
Average Skid No.	38	52	52	58	38	52	52	58
Relative Yearly Accident Benefits($)	-613	0	0	+88	-613	0	0	+88
Relative Yearly Maintenance Benefits ($)	0	0	0	0	+120	+120	+120	+120
Relative Yearly Noise Benefits($)	-	-	-	-	-	-	-	-
Total Relative Yearly Benefits ($)	-613	0	0	+88	-493	+120	+120	+208
Relative Benefit-Cost Ratio	neg	0	0	.02	neg	.04	.04	.04
Relative Yearly Benefits, Minus Costs	-5058	-4445	-4445	-4586	-1480	-867	-1029	-2557
Recommended Systems								

1. 1 = crushed stone
2 = metallurgical slag
3 = expanded slag
4 = calcined bauxite

2. 1" = 25.4 mm
1 S.Y. = 0.836 sq. m.
1 mile = 1.61 km

Table 7.9b. Benefit-Cost Tabulation for Several Conventional Resurfacing Systems

Case 2 a) 2-lane highway, each lane 12 ft wide
b) urban area
c) non-controlled access
d) 5° horizontal curve, no grade
e) heavy rainfall (zone 3)
f) ADT = 20,000
g) noise problem; however, truck % > 5%

	CRG Overlay (5" Thick) 2				Dense-Graded Bituminous Concrete Overlay (1-1/2" Thick)				Open-Graded Bituminous Concrete Overlay (3/4" Thick)				Bituminous Concrete Overlay (1-1/2" Thick) with Sprinkle Treatment				Asphalt-Rubber Seal Coat			
Aggregate Type [1]	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
Initial Cost [2] ($/S.Y.)	7.00	6.96	8.28	12.20	1.38	1.39	1.48	2.95	0.80	0.79	0.85	1.60	2.85	2.85	2.85	2.95	0.95	0.95	1.00	1.50
Initial Cost [2] ($/mi.)	98,560	97,997	116,582	171,776	19,360	19,571	20,838	41,536	11,264	11,123	11,968	22,528	40,128	40,128	40,128	41,536	13,376	13,376	14,080	21,120
Estimated Life (Years)	8	8	8	8	6	6	6	6	3	3	3	3	6	6	6	6	3	3	3	3
Yearly Cost Per Mile	18,470	18,365	21,847	32,191	4445	4494	4784	9537	4529	4473	4812	9059	9213	9213	9213	9537	5378	5378	5662	8492
Relative Yearly Cost ($)	14,025	13,920	17,402	27,746	0	49	339	5092	84	28	367	4614	4768	4768	4768	5092	933	933	1217	8492
Skid No. Range	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65
Average Skid No.	38	52	52	58	38	52	52	58	38	52	52	58	38	52	52	58	38	52	52	58
Relative Yearly Accident Benefits($)	0	+14,938	+14,938	+17,071	0	+14,938	+14,938	+17,071	0	+14,938	+14,938	+17,071	0	+14,938	+14,938	+17,071	0	+14,938	+14,938	+17,071
Relative Yearly Maintenance Benefits($)	+210	+210	+210	+210	0	0	0	0	0	0	0	0	0	0	0	0	+120	+120	+120	+120
Relative Yearly Noise Benefits($)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Total Relative Yearly Benefits ($)	+210	+15,148	+15,148	+17,281	0	+14,938	+14,938	+17,071	0	+14,938	+14,938	+17,071	0	+14,938	+14,938	+17,071	+120	+15,058	+15,058	+17,191
Relative Benefit-Cost Ratio	.01	1.09	.87	.62	std	305	44.1	3.35	0	534	40.7	3.70	0	3.13	3.13	3.35	.13	16.1	12.4	4.25
Relative Yearly Benefits Minus Costs	-13,815	+1228	-2254	-10,465	0	+14,859	+14,599	+11,979	-84	+14,910	+14,571	+12,457	-4768	+10,170	+10,170	+11,979	-813	+14,125	+13,841	+13,144
Recommended Systems						✓	✓			✓	✓							✓	✓	

1 • 1 = crushed stone
2 = metallurgical slag
3 = expanded slag
4 = calcined bauxite

2 • 1" = 25.4 mm
1 S.Y. = 0.836 sq. m.
1 mile = 1.61 km

Table 7.9c. Benefit-Cost Tabulation for Several Conventional Resurfacing Systems

Case 3
a) 2-lane highway, each lane 12 ft wide
b) urban area
c) non-controlled access
d) 5° horizontal curve, no grade
e) heavy rainfall (zone 3)
f) ADT = 20,000
g) residences are less than 10 ft from highway; truck % < 5%

	CRG Overlay (5" Thick)[2]				Dense-Graded Bituminous Concrete Overlay (1-1/2" Thick)				Open-Graded Bituminous Concrete Overlay (3/4" Thick)				Bituminous Concrete Overlay (1-1/2" Thick) with Sprinkle Treatment				Asphalt-Rubber Seal Coat			
Aggregate Type[1]	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
Initial Cost ($/S.Y.)[2]	7.00	6.96	8.28	12.20	1.38	1.39	1.48	2.95	0.80	0.79	0.85	1.60	2.85	2.85	2.85	2.95	0.95	0.95	1.00	1.50
Initial Cost ($/mi.)[2]	98,560	97,997	116,582	171,776	19,360	19,571	20,838	41,536	11,264	11,123	11,968	22,528	40,128	40,128	40,128	41,536	13,376	13,376	14,080	21,120
Estimated Life (Years)	8	8	8	8	6	6	6	6	3	3	3	3	6	6	6	6	3	3	3	3
Yearly Cost Per Mile	18,470	18,365	21,847	32,191	4445	4494	4784	9537	4529	4473	4812	9059	9213	9213	9213	9537	5378	5378	5662	8492
Relative Yearly Cost ($)	14,025	13,920	17,402	27,746	0	49	339	5092	84	28	367	4614	4768	4768	4768	5092	933	933	1217	4047
Skid No. Range	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65	30-45	4-65	40-65	50-65	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65
Average Skid No.	38	52	52	58	38	52	52	58	38	52	52	58	38	52	52	58	38	52	52	58
Relative Yearly Accident Benefits($)	0	14,938	+14,938	+17,071	0	+14,938	+14,938	+17,071	0	+14,938	+14,938	+17,071	0	+14,938	+14,938	+17,071	0	+14,938	+14,938	+17,071
Relative Yearly Maintenance Benefits($)	+210	+210	+210	+210	0	0	0	0	0	0	0	0	0	0	0	0	+120	+120	+120	+120
Relative Yearly Noise Benefits($)	0	0	0	0	0	0	0	0	+4000	+4000	+4000	+4000	0	0	0	0	0	0	0	0
Total Relative Yearly Benefits ($)	+210	+15,148	+15,148	+17,281	0	+14,938	+14,938	+17,071	+4000	+18,938	+18,938	+21,071	0	+14,938	+14,938	+17,071	+120	+15,058	+15,058	+17,191
Relative Benefit-Cost Ratio	.01	1.09	.87	.62	std	305	44.1	3.35	47.6	676	5.16	4.57	0	3.13	3.13	3.35	.13	16.1	12.4	4.25
Relative Yearly Benefits Minus Costs	-13,815	+1228	-2254	-10,465	0	+14,889	+14,599	+11979	+3916	+18,910	+18,571	+16,457	+4768	+10,170	+10,170	+11,979	-813	+14,125	+13,841	+13,144
Recommended Systems										✓	✓									

1. 1 = crushed stone
   2 = metallurgical slag
   3 = expanded slag
   4 = calcined bauxite

2. 1" = 25.4 mm
   1 S.Y. = 0.836 sq. m.
   1 mile = 1.61 km

Table 7.9d. Benefit-Cost Tabulation for Several Conventional Resurfacing Systems

Case 4 a) 2-lane highway, each lane 12 ft wide
b) rural area
c) controlled access
d) no curves, no grades
e) medium rainfall (zone 2)
f) ADT = 100
g) no noise problem

	CRC Overlay (5" Thick) 2				Dense-Graded Bituminous Concrete Overlay (1-1/2" Thick)				Open-Graded Bituminous Concrete Overlay (3/4" Thick)				Bituminous Concrete Overlay (1-1/2" Thick) with Sprinkle Treatment				Asphalt-Rubber Seal Coat			
Aggregate Type [1]	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
Initial Cost ($/S.Y.)2	7.00	6.96	8.28	12.20	1.38	1.39	1.48	2.95	0.80	0.79	0.85	1.60	2.85	2.85	2.85	2.95	0.95	0.95	1.00	1.50
Initial Cost ($/mi.) 2	98,560	97,997	116,582	171,776	19,360	19,571	20,838	41,536	11,264	11,123	11,968	22.528	40,128	40,128	40,128	41,536	13,376	13,376	14,080	21,120
Estimated Life (Years)	16	16	16	16	14	14	14	14	10	10	10	10	14	14	14	14	8	8	8	8
Yearly Cost Per Mile	12,605	12,534	14,911	21,970	2627	2656	2828	5636	1833	1810	1947	3665	5445	5445	5445	5636	2507	2507	2639	3958
Relative Yearly Cost ($)	10,795	10,724	13,101	20,160	817	846	1018	3826	23	0	137	1855	3635	3635	3635	3826	697	697	829	2148
Skid No. Range	30-24	40-65	40-65	50-65	30-45	40-64	40-65	50-65	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65
Average Skid No.	38	52	52	58	38	52	52	58	38	52	52	58	38	52	52	58	38	52	52	58
Relative Yearly Accident Benefits($)	-12	0	0	+2	+12	0	0	+2	-12	0	0	+2	-12	0	0	+2	-12	0	0	+2
Relative Yearly Maintenance Benefits($)	+105	+105	+105	+105	0	0	0	0	0	0	0	0	0	0	0	0	+60	+60	+60	+60
Relative Yearly Noise Benefits ($)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Total Relative Yearly Benefits ($)	+93	+105	+105	+107	-12	0	0	+2	-12	0	0	+2	-12	0	0	+2	+48	+60	+60	+62
Relative Benefit Cost Ratio	.01	.01	.01	.01	neg	0	0	0	neg	std	0	0	neg	0	0	0	.07	.09	.07	.03
Relative Yearly Benefits Minus Costs	-10,702	-10,619	-12,996	-20,053	-829	-846	-1018	-3824	-35	0	-137	-1853	-3647	-3635	-3635	-3824	-649	-637	-769	-2086
Recommended systems									✓	✓	✓						✓	✓	✓	

1. 1 = crushed stone
2 = metallurgical slag
3 = expanded slag
4 = calcined bauxite

2. 1" = 25.4 mm
1 S.Y. = 0.836 sq. m.
1 mile = 1.61 km

Table 7.9e. Benefit-Cost Tabulation for Several Conventional Resurfacing Systems

Case 5
a) 4-lane highway, each lane 12 ft wide
b) rural area
c) controlled access
d) no curves, no grades
e) medium rainfall (zone 2)
f) ADT = 30,000
g) no noise problem

	CRG Overlay (5" Thick) [2]				Dense-Graded Bituminous Concrete Overlay (1-1/2" Thick)				Open-Graded Bituminous Concrete Overlay (3/4" Thick)				Bituminous Concrete Overlay (1-1/2" Thick) with Sprinkle Treatment				Asphalt-Rubber Seal Coat			
Aggregate Type [1]	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
Initial Cost ($/S.Y.) [2]	7.00	6.96	8.28	12.20	1.38	1.39	1.48	2.95	0.80	0.79	0.85	1.60	2.85	2.85	2.85	2.95	0.95	0.95	1.00	1.50
Initial Cost ($/mi.) [2]	197,120	195,994	233,164	343,552	38,729	39,142	41,677	83,072	22,528	22,246	23,936	45,056	80,256	80,256	80,256	83,072	26,752	26,752	28,160	42,240
Estimated Life (Years)	8	8	8	8	6	6	6	6	3	3	3	3	6	6	6	6	3	3	3	3
Yearly Cost Per Mile	36,940	36,729	43,695	64,382	8890	8987	9569	19,073	9059	9017	9625	18,117	18,426	18,426	18,426	19,073	10,757	10,757	11,323	16,985
Relative Yearly Cost ($)	28,050	27,839	34,805	55,492	0	97	679	10,183	169	127	735	4227	9536	9536	9536	10,183	1867	1867	2433	8095
Skid No. Range	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65
Average Skid No.	38	52	52	58	38	52	52	58	38	52	52	58	38	52	52	58	38	52	52	58
Relative Yearly Accident Benefits($)	0	+3679	+3679	+4205	0	+3679	+3679	+4205	0	+3679	+3679	+4205	0	+3679	+3679	+4205	0	+3679	+3679	+4205
Relative Yearly Maintenance Benefits($)	+420	+420	+420	+420	0	0	0	0	0	0	0	0	0	0	0	0	+240	+240	+240	+240
Relative Yearly Noise Benefits ($)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Total Relative Yearly Benefits ($)	+420	+4099	+4099	+4625	0	+3679	+3679	+4205	0	+3679	+3679	+4205	0	+3679	+3679	+4205	+240	+3919	+3919	+4445
Relative Benefit-Cost Ratio	.01	.15	.12	.08	std	37.9	5.42	.41	0	29.0	5.01	.46	0	.39	.39	.41	.13	2.10	1.61	.55
Relative Yearly Benefits Minus Costs	-27,630	-23,740	-30,706	-50,867	0	+3582	+3000	-5978	-169	+3552	+2994	-5022	-9536	-5857	-5857	-5978	-1627	+2052	+1486	-3650
Recommended Systems						✓	✓			✓	✓							✓	✓	

1. 1 = crushed stone
2 = metallurgical slag
3 = expanded slag
4 = calcined bauxite

2. 1" = 25.4 mm
1 S.Y. = 0.836 sq. m.
1 mile = 1.61 km

Table 7.9f. Benefit-Cost Tabulation for Several Conventional Resurfacing Systems

Case 6 a) 4-lane highway, each lane 12 ft wide
b) urban area
c) non-controlled access
d) 3° horizontal curve, 3% grade
e) heavy rainfall (zone 3)
f) ADT = 30,000
g) residences are less than 10 ft from highway; truck % < 5%

	CRC Overlay (5" Thick)[2]				Dense-Graded Bituminous Concrete Overlay (1-1/2" Thick)				Open-Graded Bituminous Concrete Overlay (3/4" Thick)				Bituminous Concrete Overlay (1-1/2" Thick) with Sprinkle Treatment				Asphalt-Rubber Seal Coat			
Aggregate Type[1]	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
Initial Cost ($/S.Y.)[2]	7.00	6.96	8.28	12.20	1.38	1.39	1.48	2.95	0.80	0.79	0.85	1.60	2.85	2.85	2.85	2.95	0.95	0.95	1.00	1.50
Initial Cost ($/mi.)[2]	197,120	195,994	233,164	343,552	38,720	39,142	41,677	83,072	22,528	22,246	23,936	45,056	80,256	80,256	80,256	83,072	26,752	26,752	28,160	42,240
Estimated Life (Years)	8	8	8	8	6	6	6	6	3	3	3	3	6	6	6	6	3	3	3	3
Yearly Cost Per Mile	36,940	36,729	43,695	64,382	8890	8987	9569	19,073	9059	9017	9625	18,117	18,426	18,426	18,426	19,073	10,757	10,757	11,323	16,985
Relative Yearly Cost ($)	28,050	27,839	34,805	55,492	0	97	679	10,183	169	127	735	9227	9536	9536	9536	10,183	1867	1867	2433	8095
Skid No. Range	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65	30-45	40-65	40-65	50-65
Average Skid No.	38	52	52	58	38	52	52	58	38	52	52	58	38	52	52	58	38	52	52	58
Relative Yearly Accident Benefits($)	0	+30,904	+30,904	+35,320	0	+30,904	+30,904	+35,320	0	+30,904	+30,904	+35,320	0	+30,904	+30,904	+35,320	0	+30,904	+30,904	+35,320
Relative Yearly Maintenance Benefits($)	+420	+420	+420	+420	0	0	0	0	0	0	0	0	0	0	0	0	+240	+240	+240	+240
Relative Yearly Noise Benefits ($)	0	0	0	0	0	0	0	0	+4000	+4000	+4000	+4000	0	0	0	0	0	0	0	0
Total Relative Yearly Benefits ($)	+420	+31,324	+31,32	+35,740	0	+30,904	+30,904	+35,320	+4000	+34,904	+34,904	+39,320	0	+30,904	+30,904	+35,320	+240	+31,144	+31,144	+35,560
Relative Benefit-Cost Ratio	.01	1.13	.90	.64	std	319	45.5	3.47	23.7	275	47.5	4.26	0	3.24	3.24	3.47	.13	16.7	12.8	4.39
Relative Yearly Benefits Minus Costs	-27,630	+3485	-3481	-19,752	0	+30,807	+30,225	+25,137	+3831	+34,777	+34,169	+30,093	-9536	+21,368	+21,368	+25,137	-1627	+29,277	+28,711	+27,465
Recommended Systems						✓	✓			✓	✓									

1. 1 = crushed stone
2 = metallurgical slag
3 = expanded slag
4 = calcined bauxite

2. 1" = 25.4 mm
1 S.Y. = 0.836 sq. m.
1 mile = 1.61 km

weight is taken as 1500 lb/yd^3 (900 kg/m^3). For the bituminous concrete overlay with sprinkle treatment, it is assumed that the coarse aggregate in the overlay is always crushed stone and that it is the sprinkled aggregate which is made to vary; the application rate of the sprinkled aggregate is 5 lbs/yd^2 (2.7 kg/m^2). The fine aggregate used in the mix in all cases is assumed to be natural sand.

The yearly cost per mile (.62 km) for each resurfacing system can be obtained in a manner similar to that shown below. The calculations in this example are for a 1.5 in. (3.8 cm) thick dense graded bituminous concrete overlay using expanded slag coarse aggregate and being governed by the conditions of Table 7.9b:

Average cost of dense-graded bituminous concrete overlay 1.5 in. (3.8 cm)	= $\$1.375/yd^2$ ($\$1.64/m^2$) (See Table 7.4)

The above cost assumes typical crushed stone aggregate (about \$2.50/ton, \$2.75/Mg, see Table 7.2). Expanded slag aggregate is about \$6.00/ton, \$6.60/Mg, (Table 7.2). Thus, the cost of an expanded slag overlay is an additional \$3.50/ton (\$3.90/Mg) of coarse aggregate. Since it has been assumed that 1500 lb/yd^3 (900 kg/m^3), the extra cost is $\$3.50 \times \frac{1500}{2000} \simeq \$2.63/yd^3$ ($3.46/m^3$). Since an area of 1 yd^2 (.84 m^2) with a thickness of 1.5 in. (3.8 cm) is $\frac{1}{24}$ yd^3 ($\frac{1}{28.5}$ m^3), the extra cost is $\frac{\$2.63}{24} \simeq \$0.11/yd^2$ ($\$0.13/m^3$). Thus, the estimated initial cost of the overlay using expanded aggregate is $\$1.375 + \$0.11 \simeq \$1.48/yd^2$ ($\$1.76/m^2$). Multiplying $\$1.48/yd^2$ ($1.76/m^2$) by 14,080 yd^2 (16760 m^2) (i.e., the area of a one-mile (1.61 km) stretch of 2-lane highway, each lane 12 ft. (3.6m) wide), the initial cost becomes \$20,838/mi (12,943/km).

An estimate is made for the expected life of the resurfacing system--six years in this case. The estimate is dependent on the traffic and geographical conditions as well as the resurfacing system.

The yearly cost per mile (1.61 km) is then computed by using the formula:

$$YC = TC \times CRF\ (i,\ n)$$

where

YC = yearly cost

TC = total cost

CRF = capital recovery factor

i = interest rate

n = estimated life

Assuming an interest rate of 10%, YC = \$20,838 x .2296 = \$4784.

After the yearly cost is determined for each of the resurfacing systems being evaluated, the system having the lowest yearly cost is selected. This system becomes the standard on which incremental or relative costs and benefits are based. Since in Table 7.9b the dense graded bituminous concrete overlay with type 1 aggregate (crushed stone) has the lowest yearly cost (\$4445/mile or \$2760/km), it becomes the standard. Thus, the yearly cost of each system relative to the standard is that system's yearly cost minus \$4445. In the case of the dense-graded expanded slag-bituminous concrete overlay, the relative yearly cost is \$4784 - \$4445 = \$339.

The benefit models are based mainly upon the skid-resistance characteristics of the resurfacing systems. The skid-number range during the lifetime of the system must first be estimated. This range is dependent upon the resurfacing system (especially the aggregate used), the life of the system, and traffic and geographical conditions. In Tables 7.9a through 7.9f, an average SN is computed from the estimated range of skid numbers. This average SN becomes the independent variable which is used to define the yearly accident, noise, and tire-wear benefits (optional).

The yearly benefits of the dense-graded, expanded slag overlay can now be calculated by using the benefit models as shown below:

Standard system average SN = 38

Dense-graded expanded slag overlay average SN = 52

Difference in SN = 14

Relative yearly accident benefits:

(According to Figure 7.2, when SN is less than 45, each unit increase in SN results in a 2.0 decrease in wet-accident rate; when SN is more than 45, the decrease in rate is 0.4.)

$$\text{Avg. decrease in rate} = \frac{(2.0)7 + (0.4)7}{14} \times 17.4 \left(\begin{matrix}\text{correction}\\ \text{factor}\end{matrix}\right) = 20.88$$

$$\text{Savings} = 20.88 \text{ accidents per SN} \times 14 \times 20{,}000 \frac{\text{vehicles}}{\text{day}}$$

$$\times\ 365 \frac{\text{days}}{\text{year}} \times \$700 \text{ per accident} = \$14{,}938$$

Relative yearly noise benefits:

0 (Effect of pavement texture on noise is negligible because of high truck percentage.)

Relative yearly tire-wear benefits:

Optional model; not included in Table 7.9.

Relative yearly maintenance benefits:

It is assumed here that the yearly maintenance requirements for the dense-graded, bituminous concrete overlay being evaluated are the same as for the standard system; therefore, yearly maintenance benefits = 0. (In the case of the asphalt-rubber seal system, however, it is assumed that 800 ft (20 m) less of crack cleaning and sealing per year is required in comparison to the standard system; thus, labor and equipment costs = \$40, material costs = \$10/lf x 800 lf = \$80, giving a benefit of \$120.)

Total relative yearly benefits:

\$14,938 + 0 = \$14,938

Now, since the relative yearly cost of the dense-graded, expanded slag, bituminous concrete overlay is \$339, and the relative yearly benefits are \$14,938, the relative benefit-cost ratio is $\frac{\$14,938}{\$339} = 44.1$.

Because of its conciseness, the benefit-cost ratio is a frequently used index. Generally, a ratio greater than one means that the alternative is a desirable one. However, this ratio should not be the only basis for comparison because it fails to take into account the magnitude of the costs and benefits and may, therefore, be misleading. For instance, had the relative yearly cost been \$1.00 and the relative yearly benefits \$44.10, the result would have again been a benefit-cost ratio of 44.1. However, because the magnitude of the costs and benefits is so small in the latter case, it would be absurd to claim that one system is superior over the other. Thus, the benefit-cost evaluation of the dense-graded, expanded slag, bituminous concrete overlay should be viewed in the following manner. If one spends \$339 yearly above the cost of the standard system in order to obtain the desired system, the subsequent yearly benefits would increase by \$14,938. It is therefore obvious that the dense-graded, expanded slag overlay should be recommended (the recommended systems in Table 7.9a through 7.9f are indicated by a check mark [✓]).

There may, of course, be cases where the decision is not so obvious. For instance, a resurfacing system may have a relative benefit-cost ratio much greater than 1, but budget limitations may prevent the recommendation of that system if the additional yearly cost is high. Or, at the other extreme, a resurfacing system's relative benefit-cost ratio may be negative, but because the system is economically similar (on an absolute scale) to a standard system which is being recommended, it may be that it should not be excluded. Still another consideration which should enter the decision process at this time is the influence of non-quantifiable benefits. These benefits may be large enough to cause the decision-maker to accept a seemingly uneconomical resurfacing system (or, alternately, to reject a seemingly economical system). Thus, the factors involved in deciding which resurfacing system(s) to recommend should include the relative benefit-cost ratio, the magnitude of the costs and benefits, any non-quantifiable benefits, any imposed budgetary constraints, and intuition. The recommended systems in Tables 7.9a through 7.9f have been selected through such considerations.

A comprehensive examination of Tables 7.9a through 7.9f yields the following observations:

1. The open-graded, bituminous concrete overlay is a recommended system under each of the conditions analyzed.
2. The aggregates most frequently recommended are metallurgical slag and expanded slag. Calcined bauxite has cost disadvantages; crushed stone does not provide lasting skid resistance. Crushed stone, however, may be a desirable aggregate for use in low ADT, low-accident roads (see Table 7.9d).
3. Dense-graded, bituminous concrete overlays (with skid-resistant aggregates) offer a possible alternative for high ADT roads.

4. Asphalt-rubber seal coats are recommended under some conditions. Their primary advantage is low cost; however, their expected service life is often low. Thus, they are not recommended where resurfacings must be kept to a minimum.
5. As expected, the continuously reinforced concrete overlay is not recommended under any of the given conditions. This is because the system is more expensive, being much thicker than the other systems, yet its yearly benefits are approximately the same. The life of the continuously reinforced concrete overlay is generally greater than the life of the other systems, and it may be that such a system would be economic in locations where frequent resurfacings are undesirable. Also, two-inch (50 mm), non-reinforced bonded concrete overlays have been recently placed and may prove to be economical in some cases.
6. The bituminous concrete,sprinkle treatment overlay is not generally recommended for any of the given conditions. While the sprinkle treatment method is shown as being more costly than the dense-graded and open-graded overlays and the asphalt-rubber seal coat, it must be kept in mind that costs associated with the long-distance transportation of aggregates cannot be reflected in a general economic model such as this one. Thus, it may be that the use of sprinkle treatments is more economical in certain situations involving the long-distance transportation of aggregates, for example, in areas which have a limited availability of skid-resistant aggregate.
7. The overall benefit-cost model is not extremely sensitive to changes in assumed unit costs. In other words, varying the unit

costs will not have a great effect on the final recommendations For example, Tables 7.9a through 7.9f were recalculated using the unit cost of an urban accident as $1500 and the unit cost of a rural accident as $4000. (It had been $700 and $2000, respectively.) The systems chosen for recommendation did not change appreciably.

8. The overall benefit-cost model is more sensitive to cost, expected service life, and skid-number estimates. The systems recommended in Tables 7.9a through 7.9f may therefore be narrowed down by varying the estimates within the range of values.

7.5 Economic Analysis Summary

The procedure that has been described is intended as a method which can be used to evaluate the appropriateness of spending research dollars on new resurfacing systems that are being or will be introduced. With this procedure, benefit-cost tabulations of the new system can be performed in the manner indicated and compared with the results of conventional systems. The new systems can thus be eliminated if they cannot be justified in any application under various representative conditions. It may also be that the new systems are justifiable only under a few extreme conditions or localities, which may make any further research or development effort unnecessary.

The overall benefit-cost model used for the analysis comprises several component models: a noise model, an accident model, a tire wear model, etc. The component models are merely skeleton models which have been greatly simplified to make the analysis manageable. (In some cases, the data are not presently available to obtain a more sophisticated model.) The component models can be made more accurate by taking more variables into account; but, at the same time, the overall model becomes less manageable because more specific conditions (roadway, traffic, climate, etc.) have to be described, thereby detracting from the generality which is desired. The individual component models can also be easily modified to suit the user's preferences.

Finally, it cannot be overemphasized that care must be taken in using the benefit-cost model discussed here. The model is applicable for general situations and should not be used for the more frequent situations in which it is desired to establish the most economical system for application at a given location. The benefit-cost model does not consider such costs as that of hauling aggregates for long distances or the costs of having to reroute traffic during construction or maintenance operations. This is because the specific circumstances must be known before these costs can be determined. The benefit-cost ratio appears to be as much a function of specific traffic and environment conditions as of the resurfacing system. Thus, the systems which are recommended must be qualified with a statement indicating the specific conditions under which the recommendation is applicable.

8.0 SUMMARY OF RESULTS AND RECOMMENDATIONS

As a result of the literature review and a limited laboratory and field study, a number of results have been established. Based on these results, a variety of suggestions for future research have been developed. In this section the results obtained in the research are first described, and then followed by suggestions for additional research.

8.1 Summary of Results

A significant finding of the research was the development of a procedure for predicting field skid numbers (SN) from laboratory data. The research shows that the skid number (SN) can be predicted from the root mean square height of the macrotexture (RMSH) and BPN:

$$SN_V = (1.38\ BPN - 31)\ e^{-.038V\ RMSH^{-.52}} \qquad 2.9c$$

An alternative prediction can be obtained by relating skid number to BPN and Sandpatch Mean Texture Depth (MD):

$$SN_V = (1.38\ BPN - 31)\ e^{-.06V\ MD^{-.52}} \qquad 2.9d$$

It is important to note that the relationships do not take into account the influence of seasonal variations. However, laboratory methods can be used to estimate skid numbers at any speed from representative samples of the pavement surface. In designing aggregates for good skid resistance at high speeds, it is important to provide large values of root mean square texture height (or mean texture depth). For aggregates which polish to a given BPN or to a known minimum RMSH 2000 microtexture, it is possible to compute the macrotexture that is required to provide the desired level of skid resistance at the design speed.

8.1.2 Aggregate Role and Most Important Properties

The role of the aggregate is to provide, throughout the life of the pavement surface, a microtexture that will maintain a minimum level of friction under prevailing conditions, thus eliminating or reducing unsafe skidding on wet surfaces. In bituminous surfaces, the aggregate must also provide a macrotexture that will induce tire hysteresis and facilitate water removal from the tire-pavement contact area, thereby inhibiting the buildup of a thick water film which could partially separate the tire from the pavement surface.

The most important properties that affect aggregate performance are:

1. Hardness and differential hardness of constituent minerals.
2. Proportion of hard to soft fraction of constituent minerals.
3. Grain consolidation and bonding in the matrix.
4. Size, shape and distribution of the hard crystals in the less hard matrix.
5. Strength and resistance to weathering through detrimental physical agents and/or chemical effects.
6. Shape, size and distribution of aggregate particles in the surface mixture.

In short, the most important properties are those that make the aggregate both skid-resistant and wear-resistant throughout the expected surface life. Performance properties that may alleviate noise generation, tire wear, light reflection, and other performance requirements are also important and are mentioned under the respective performance requirements.

8.1.3 Skid Resistance and Tire Noise

Pavement-tire noise is largely a function of tire properties and tire tread design. However, the type of pavement surface has been found to influence tire noise. Despite the lack of adequate models to describe the mechanism of noise generation, some tentative but plausible inferences may be drawn about the selection and use of aggregates for quieter pavements. Significantly, the objectives of quieter pavements and pavements with adequate skid resistance are not diametrically opposed.

Improved noise performance has been obtained with open graded plant mix seals when compared to conventional dense graded bituminous mixtures. Normally, these mixtures contain angular aggregates having coarse (skid-resistant) texture.

On PCC surfaces, surfaces with pronounced transverse markings were found noisier than smoother textured surfaces. Worn PCC, where the coarse aggregate is exposed, were found considerably noisier than new, relatively smooth PCC surfaces.

8.1.4 Other Factors Related to Aggregate Properties

Tire wear was found to be mainly a function of tire characteristics rather than of aggregate or pavement properties. However, tire slippage and increasing microtexture were found to increase tire wear. Only limited quantitative data are available on tire wear due to aggregate or surface roughness.

The effect of aggregate on rolling resistance and power consumption was found to be small, although at extreme macrotexture levels the effect is significant. Power consumption of pneumatic tires is comparable to the power consumed by vehicle components such as the muffler, air cleaner, and emission control devices - on the order of 5 to 10 percent of the total power losses at operating speeds. It was found that the role played by the type of tire was more influential in determining rolling resistance than the type of aggregate.

Light-colored aggregates having reflective properties that result in a uniform, non-glare, pleasing color are desirable. On the other hand, those that may cause reflection that results in vivid shiny surfaces or surface glare should not be used in pavement surfaces. Well-textured surfaces that provide good skid resistance at normal traffic speeds also reduce glare, splash and spray.

In bituminous mixtures, it is important that the aggregate withstand plant mixing temperatures of 300-400° F (150-200° C) without exhibiting decomposition, weakening or any other deleterious changes. In any mixture, the surface aggregates must be stable and durable for the design life of the surface when exposed to environmental conditions and maintenance practices, such as freezing and thawing, wetting and drying, salt deicing, and other possible effects due to weathering.

8.1.5 Optimal Pavement Surfaces for Different Situations

Warrants for optimal surfaces depend highly on the level of required performance, as discussed earlier in Section 4.8. Four performance pavement, site categories were listed: Very Difficult, Difficult, Average, and Easy Sections.

In the "Easy" performance category (e.g., lightly traveled roads, driveways, and minor residential streets), no problems are anticipated whatever the type of pavement or aggregates used, as long as they meet commonly used conventional specifications and construction practices. For example, if the most economical aggregate available is limestone or dolomite, it should be satisfactory for light traffic when it is used without blending, and for medium and heavy traffic when either blended with higher quality aggregate or when the fine aggregate is of a better quality. On the other hand, using excess asphalt in the mix, which may cause bleeding, is not satisfactory for any use. In the "Average" category (most city streets and rural roads where no steep grades, sharp curves, or unusual hazardous locations exist), most conventional surfaces will be satisfactory if conventional specifications and construction practices are carefully observed. However, in this category, the aggregate must be of at least medium grade with a history of proven performance, such as granite, gneiss, carbonate aggregates with high sand-size silicious content, and some slates and shales.

In the "Difficult" performance category, which includes heavily traveled high speed highways and steep grades and sharp curves on "Average" highways, the best available conventional designs (e.g., high type PCC or high type dense or open-graded asphalt plant-mixes) should be utilized in conjunction with careful observation of construction practices. In this category, only high quality natural or synthetic aggregate with a proven record of performance should be used, such as selected crushed sandstone, arkose, graywacke, high friction quartzite or slag, and durable, crush-resistant lightweight aggregate such as some expanded shales, clays and slates. Where these materials are not economically available, high-type proven igneous and metamorphic aggregates such as some basalt, granite, and gneiss may be used. In this category, it is also recommended that natural silica sand or a similar material that performs well be used for the fine aggregate portion.

Finally, in the "Very Difficult" category, which includes curves on high-speed highways, approaches to toll gates, heavily trafficked intersections, pedestrian crosswalks, and steep downgrades and lane-change locations on high speed highways, only the "best" designs and highest quality aggregate and well-controlled construction practices will result in optimum pavement surfaces. Aggregates for these locations must be of the highest performing, naturally occurring or synthetic types such as proven high-quality wear-resistant and skid-resistant sandstone, graywacke, and quartzite, or when possible, emery or synthetic aggregates that have been proven to be of superior quality, such as calcined bauxite, calcined flint and some ceramic materials. One alternative for Portland cement concrete may be to add aluminum oxide or silicon carbide granules of a medium-size gradation (#16 to #100 sieve) to the top 1/4-to 1/2 in. (6 to 13 mm) of the plastic concrete during placement. More experience is needed to prove the effectiveness of this procedure. It is also recommended that high strength binders be used in the "Very Difficult" category to hold the high quality aggregate particles in place for reasonably long periods under continuous, high stress conditions. More research is needed in the "Very Difficult" performance category with the objective of finding more innovative aggregates and surfacing systems that provide long-lasting surfaces at economical costs. For the purpose of developing new, high performing, wear-resistant and polish-resistant aggregates, target values for the necessary aggregate properties are presented in Table 3.7. These values are based on high but conceivably attainable goals as reported in the literature.

It must be remembered that in any of the above performance categories a variety of other requirements must also be met, depending on the given situation. For example, where noise abatement is of high priority, friction requirements associated with high levels of macrotexture, coarse mortar texturing, or transverse grooving may have to be compromised, possibly requiring a lowering of the traffic speed. Lowering the traffic speed may also be required if reduced tire wear and fuel consumption are critical requirements.

8.1.6 Aggregate Evaluation

The following procedure is suggested for evaluating aggregates that do not have a known performance record:

1. (a) Find the Los Angeles Abrasion loss for naturally-occurring normal-weight aggregates (sp. gr. 2.5 to 3.0). L.A. loss should not exceed 40 percent and should preferably be much less (25 percent or less) for aggregates to be used in "Difficult" or "Very Difficult" pavement sections, or

(b) Find Aggregate Abrasion Value (AAV), Aggregate Impact Value (AIV), and Aggregate Crushing Value (ACV), using British procedures (BS 812-75) or similar procedures when available, particularly if the aggregate does not lend itself to the Los Angeles test, as in the case of some synthetic aggregates. For "Difficult" and "Very Difficult" sections, values of AAV, AIV, and ACV should not exceed 10, 20 and 20 respectively. In all three cases, lower values imply better performance. Target values not exceeding half of the allowable values are recommended for very high-performance surface aggregates.

2. Perform a laboratory polishing test that has been correlated with aggregate performance in service, e.g., British Wheel Test or any other polishing test. Polished Stone Value (PSV) or similar values should be no less than 65 British Portable Number (BPN) units and preferably 70 units or higher for "Difficult" and "Very Difficult" sections.
3. Perform a petrographic analysis in addition to other routine weathering-related tests, and determine the susceptibility of constituent minerals to known or suspected chemical reactions and detrimental physical influences (e.g., wetting and drying, freezing and thawing). Also, the porosity of the aggregate should be determined and must not exceed 35 percent of aggregate volume. For sections where skidding may be a major problem (as in difficult sections), porosity in the range of 25 to 35 percent is recommended. Porosity is not to be confused with water absorption which generally should not exceed about 2 to 3 percent--the lower, the better.
4. Perform tests to check aggregate susceptibility to weathering, especially if need for such tests is indicated by the petrographic analysis. Freeze-thaw tests and those that may in-

dicate suspected chemical reactions should receive priority consideration in checking for weathering effects.

8.1.7. Selection of Synthetic Aggregates

A wide variety of approaches are possible for the production of synthetic or man-made skid-resistant aggregates. Manufacturing techniques, raw materials, and aggregate properties are interrelated, so that one cannot be specified to the exclusion of the other. The following scheme is suggested for selecting or producing new synthetic aggregates:

1. Determine chemical composition of raw materials to evaluate potential for growth of hard crystals, (e.g., potential for producing α - alumina).
2. Consider modifying the raw material by fluxing, doping with harder material (e.g., doping with calcined bauxite) or other means.
3. Determine optimal aggregate type for a particular material to be produced.
4. Select a manufacturing process compatible with material and aggregate type (e.g., sintering).
5. Perform economic analysis of proposed system considering energy costs, capital outlay, market area, production capacity, etc.
6. Produce material; bench scale and pilot plant scale.
7. Perform laboratory wear and polish tests along with other conventional aggregate tests.
8. Perform field demonstration study based on pilot plant material.
9. Release to market place for commercial development.

Sintered materials such as clay, shale, and slate appear to be the most viable in the short run. More use could be made of non-bloating shales and clays, particularly if they could be doped with hard, fine materials such as crushed bauxite or quartz sand. Their wear resistance and applicability to northern climates need to be verified.

Alumina-based materials such as calcined bauxite, sintered red muds and high alumina clays appear promising. A detailed search for raw materials and a laboratory scale development for these materials should be initiated to promote commercial development.

Based on British literature, refractory brick materials appear promising. Current methods of brick manufacture are inefficient (require crushing and sizing), and developmental work should be initiated to determine ways to efficiently transfer brick manufacture technology to the manufacture of aggregates.

Beneficiation should also be pursued. While the applicability of current material specifications may need to be reviewed relative to surface aggregates, clearly there are many aggregates with marginal durability but excellent skid resistance (e.g., arkose and other porous sandstones). Impregnation of polymers or inorganic cements should be evaluated with the objective of improving both the environmental and mechanical durability of these materials. Beneficiation for the purpose of increasing skid resistance, e.g., by thermal treatment or coatings, does not appear to be of high priority from an economic standpoint.

8.1.8 Pavement Systems

Of the surfacing systems considered, the following have been most commonly and successfully used to date, and may be improved as the quality of aggregate and binder improves.

1. Dense-graded asphalt concrete, high quality aggregates, binder and mix design.
2. Open-graded asphalt concrete, hot plant mix, high quality aggregates, binder and mix design.
3. Epoxy-asphalt seal coat using high quality aggregate (e.g., Calcined Bauxite).
4. Portland cement concrete, high strength mix design, textured with tines or combs.
5. Sawed transverse or longitudinal grooving of PCC surfaces.

Systems 1 through 3 may be applied to newly constructed surfaces or to rememdy worn and/or polished surfaces of either bituminous of PCC pavements. System 4 is recommended only for new pavements and system 5 is recommended only as a remedial procedure for PCC surfaces.

8.1.9. Cost Benefit Analysis

1. Tentatively, the benefit (negative) associated with tire wear has a relatively large magnitude. This is because tire-wear costs, although small when considering a single vehicle, are considerable when accumulated for all drivers.
2. The benefits associated with accidents and noise have a large magnitude in some cases. Accident and noise costs may be considerable on an individual basis, but accidents affect only the people who are involved, and noise affects only a few residents under certain conditions.
3. The benefit associated with maintenance appears to be a secondary one. Most resurfacing systems appear to require about the same amount of maintenance (true, at least, for conventional systems).

4. System costs vary more widely than system benefits. The cost of a system may be the only consideration necessary in making a decision. A high cost is a large disadvantage, because the total benefits would have to be tremendous in order to balance the costs. Total benefits are not usually large because individual benefits often cancel each other out (i.e., if accident benefits are positive, the noise and tire-wear benefits are probably negative).
5. The overall benefit-cost model is not highly sensitive to changes in assumed unit costs. In other words, varying the unit costs will not have a great effect on the final recommendations.

8.2 Recommended Research

Based on the literature review and a limited laboratory and field study a number of areas that warrant additional research or development have been identified. In the following sections each of these areas are briefly described along with a listing of the tasks required to complete the research.

8.2.1 The Relationship Between Pavement Texture and Skid Resistance

Objective: Refine the prediction model for the skid resistance versus speed resulting from pavement microtexture and macrotexture.

The investigators have established that a relationship exists between skid resistance at any speed and pavement texture. It is the purpose of this proposed research to refine this relationship and to increase the data base upon which it is based. The model appears promising and is consistent with observations in the literature. Therefore, it has a high probability of success. Also included in this effort should be a streamlining of data processing in order to facilitate implementation. The required tasks and estimated cost are:

Task 1. Verify the model (equation 2.4) for the skid resistance-speed behavior for at least 25 actual surfaces having a wide range of texture.

Task 2. Obtain microtexture and macrotexture data for these surfaces and examine their correlations with SN_o and PSNG in the model.

Task 3. Select the best microtexture and macrotexture parameters and develop methods for their direct measurement.

Cost Estimate	$130,000

8.2.2 Shales - Slates - Clays as Skid Resistant Aggregates

Objective: To expand the use of sintered clays, shales, and slates as skid resistant aggregates.

Sintered materials produced from shale, clay, and slate have shown promise as skid resistant aggregate. The associated technology and materials are those associated with sintered lightweight aggregate. The advantage of these materials is that currently available technology and equipment can be readily adapted to their manufacture.

Additional research is needed on two fronts. First, the attractiveness of sintered shales, slates and clays would be enhanced if their wear resistances could be improved. This could be done by varying the proportions of the raw materials and the degree of bloating. This may violate the specifications for structural lightweight aggregate and the consequences of market availability for the nonspecification material should be evaluated. Cost of these materials should be in the vicinity of $15 per ton.

Secondly, more field experience (with regard to both construction and durability) is needed in northern climates. A field demonstration project using standard lightweight as well as lightweights or non-bloated aggregates as described above in a variety of wearing courses is suggested. This would provide a basis for extended use of sintered materials in the north where they have been little used. The required tasks and estimated cost are:

Task 1. Perform state of art review of field and manufacturing experience with bloated and non-bloated sintered shales, clays, and slates.

Task 2. Identify techniques to improve wear and polish resistance of expanded clay, shale, and slate.

Task 3. Establish optimum aggregate properties for skid and wear resistance.

Task 4. Perform economic analysis for more promising of the aggregates to include factors such as plant capacity, alternate uses of aggregate, service life, shipping costs, etc.

Task 5. Conduct demonstration project in various parts of country using different aggregates and different pavement systems.

Cost Estimate $150,000

8.2.3 Improved Methods for Evaluating Wear and Polish Resistance

Objective: To identify the basic mechanism of wear and polish and, based on these mechanisms, to develop rational laboratory test procedures to evaluate aggregate wear and polish resistance.

The wear and skid resistance of the pavement surface is determined in large part by the aggregate used in the pavement. In spite of the importance of the role played by the aggregate there are no generally accepted methods that can be used to predict the wear or friction resistance of aggregates when they are exposed to the cumulative effects of traffic and the environment. The present state of the art relies mostly on empirical test procedures such as the Los Angeles abrasion test. Considerable success has been obtained with the British wheel (ASTM D 3319); however, the test was designed to measure polishability and not wear. Further, most of the test methods and test criteria were developed for conventional aggregates and they are very difficult to extrapolate to non-conventional aggregates such as expanded or composite aggregates.

As well, little is known about the actual mechanisms of polish and wear. An understanding of <u>how</u> aggregates wear and <u>how</u> they polish is essential to the development of rational test methods for measuring polish and wear.

Therefore, better test methods as well as a better understanding of the mechanisms of wear and polish would appear to be prerequisites to the rational, orderly, development of new, improved, sources of skid and wear resistant aggregates. This is particularly true because the factors that improve skid (or polish) resistance often accelerate aggregate wear. Sophisticated aggregates of the future must provide a balance between wear and skid resistance.

It is recommended that a research program be initiated that would first investigate the basic mechanisms of wear and polish resistance and secondly, based on a better definition of the mechanisms, develop a rational set of test procedures to evaluate the potential wear and polish resistance of aggregates. The required tasks and cost estimate are:

Task 1. Conduct a thorough literature search on the mechanism of wear and polish as related to pavement aggregates.

Task 2. Identify the more promising mechanistic models useful in describing the phenomenon of wear and polish given different aggregate types.

Task 3. Modify existing wear and polishing test procedures, or develop new procedures, to verify the proposed models from Task 2.

Task 4. Recommend an optimum test procedure for evaluating aggregate wear and polish.

Task 5. Verify the procedure of Task 4 by field experimentation.

Estimated Cost $175,000

8.2.4 Basic Aggregate Properties

Objective: Identify and characterize through laboratory testing the basic aggregate properties that contribute to and control the wear and polish of aggregates such that wear and polish resistance can be predicted from basic aggregate properties.

While considerable research has been done in an attempt to relate aggregate and surface properties to skid resistance, an effort should be made to relate the more basic aggregate properties to their wear and skid resistance characteristics. The question to be answered: Can skid resistance and wear resistance be predicted with a reasonable degree of accuracy and confidence from the basic aggregate properties such as petrography, aggregate texture, shape, size, permeable voids, rate of wear, etc. The research should cover a wide range of natural and synthetic aggregates used across the United States, and should take into consideration such variables as traffic counts, loads, and speeds, and environmental influences. The required tasks and estimated cost are:

Task 1. Quantify basic aggregate properties that are known to influence or suspected of influencing wear and polishing.

Task 2. Categorize aggregates of known performance and relate their basic properties to their performance through laboratory testing.

Task 3. Verify Task 2 through field testing.

Task 4. Determine measures to correct causes of wear and polishing for aggregates where it is economically feasible and recommend most feasible and economical use of different aggregates-aggregate combinations.

Task 5. Develop a set of basic aggregate properties that are necessary for different aggregates according to their use in different performance categories.

Cost Estimate $200,000

8.2.5 Brick Manufacture Technology Related to Skid Resistant Aggregates

Objective: To modify and extend basic brick technology for use in manufacturing skid resistant aggregates.

The British studies have shown that many bricks, when crushed, can give excellent skid resistance along with excellent wear resistance. Further, these materials do not develop their skid resistance at the expense of accelerated wear as with most bloated aggregates. Some of the bricks evidence poor durability from environmental effects, principally from water and/or freezing and thawing.

Brick manufacture is not particularly efficient for processing large volumes of material as required in the manufacture of aggregates. Furthermore, the bricks would have to be crushed to size causing a waste of fines and considerable crushing cost realizing the hardness and abrasiveness of a highly skid resistant aggregate. Ideally, the aggregate should be fired to size and not crushed to size.

Brick technology, especially that of refractory bricks, involves liquid phase sintering (Table 5.4). Bloating is traditionally not involved but, as with molochite, Table 5.8, there is no reason that some bloating cannot be specified as long as adequate wear is retained. The scope of the proposed research would investigate materials and processes that can be used to produce tailor made liquid phase sintered aggregates, especially materials and processes borrowed from brick technology. The required tasks and estimated cost are:

Task 1. Review the current state of the art regarding materials and manufacturing processes involved in common and refractory brick manufacture.

Task 2. Identify a set of target aggregate properties that are suited to aggregates made according to brick manufacture.

Task 3. Recommend modifications in materials, processes, and target specifications based on a review of Tasks 1 and 2. It is anticipated that process changes and raw material changes may be warranted given the quantity of material required and the altered requirements for pavements as compared to say refractory brick applications.

Task 4. Produce material on pilot plant or laboratory bench scale.

Task 5. Evaluate material using laboratory wear and polish testing.

Task 6. Verify Tasks 4 and 5 through pilot plant production and field wear and polish testing.

Cost Estimate $350,000

8.2.6 Beneficiation of Aggregates Having Good Skid Resistance But Poor Durability

Objective: To identify technically and economically sound methods of upgrading marginal aggregates that possess good skid resistance but poor durability.

There are a number of aggregates that possess good or excellent skid resistance but that exhibit poor durability. Examples are some sandstones and some gneisses that degrade under the effect of traffic and/or the environment. To improve the durability of these aggregates would result in very significant sources of aggregates, often located in areas where there are not acceptable natural aggregates.

Exact mechanisms of beneficiation have not been established but the most viable appears to be some sort of polymer impregnation. The polymer would fill any voids and additionally could provide mechanical cementing strength between the individual grains within the aggregate particles. The technology of polymer impregnation is well understood and would not require extensive research and development.

Alternative techniques for beneficiation should also be investigated according to the deficiency that requires upgrading. For example, some of the more porous aggregates may also require some sort of chemical treatment to minimize absorption. The required tasks and cost estimate are:

Task 1. Identify aggregates that have adequate skid resistance but poor durability.

Task 2. Identify specific aggregate characteristics that require upgrading.

Task 3. Perform upgrading procedure on selected aggregates.

Task 4. Perform laboratory testing to verify skid and wear resistance and verify effectiveness of upgrading techniques.

Task 5. Recommend upgrading processes for various classes of marginal aggregates.

Cost Estimate $100,000

8.2.7 Mixture Design Procedure to Allow for Surface Texture

Objective: Develop a mixture design procedure that incorporates mixture surface texture as a design parameter.

Prior research has shown that a relationship exists between skid resistance and pavement texture. Recently, open graded friction course mixtures have been developed to provide a high level of surface texture and special procedures have been developed to design these mixtures. Conventional wearing courses are designed by a variety of methods depending on the particular agency. However, in none of these design procedures is the question of surface texture addressed explicitly. Limits on mixture gradation or mixture voids are often specified but these are implicit requirements as far as surface texture is concerned.

Prior research has shown that the BPN and the measured macrotexture can be used reliably to compute the skid resistance at different speeds. The BPN is basically a measure of aggregate microtexture and its terminal value can be estimated from accelerated laboratory testing. Macrotexture is determined by gradation, compaction, binder properties, and aggregate wear.

A laboratory procedure is needed that would permit the designer of bituminous mixtures to design for a specified terminal level of skid resistance obtained in the field under the action of traffic and weather. This is possible if the design of bituminous mixtures can be modified to include a projected level of macrotexture. Then, by including a BPN measurement, it will be possible to design the bituminous mixture to provide, not only durability and stability but also, a level of terminal skid resistance at the desired design speed. A research program should be initiated to develop a method of mixture design that will allow for the prediction of surface texture. The procedure should consider wear, and environmental factors, as well as compaction due to both construction and traffic.

The required tasks and cost estimate are:

Task 1. Review the literature and current practice to select a method of preparing laboratory specimens that reproduce texture produced in field construction.

Task 2. Select a test procedure that reproduces the effect of construction, traffic and the environment with respect to polish and wear.

Task 3. Determine the effects of aggregate gradation and other aggregate properties on macrotexture.

Task 4. Determine the effects of binder properties on pavement macrotexture.

Task 5. Determine the effects of binder/aggregate ratio on pavement macrotexture and establish the degree of control possible on macrotexture by mix design.

Task 6. Determine the effects of blending of aggregate types on macrotexture.

Cost Estimate $150,000

8.2.8 Tire Pavement Interaction Noise

Objective: Develop a model that explains tire-pavement interaction noise and specify a design for a low noise pavement texture.

The investigators believe that the research tasks enumerated below will show why tires are quieter on some pavements than on others, and will contribute to the development of methods for designing pavements (including aggregate selection) so as to reduce tire-pavement interaction noise.

It is recommended, however, that a low priority be set for accomplishing these tasks within the program of overall aggregate research. Other agencies such as the Environmental Protection Agency now have ongoing programs in tire noise modeling, and the role of pavement texture in the noise generating process is already receiving attention. If noise considerations continue to be a part of the aggregate research program, however, the approach presented below should be considered.

Note that the order of undertaking of the noise tasks should be that in which they are listed, for the results of the early tasks are required in performing those listed later. Further, it should be remarked that much of the work set out in the tasks is new ground. The probability of success of the final tasks hinges on the successful accomplishment of the earlier ones and the series should be undertaken in closely monitored steps.

Task 1. Determine the tread vibration response to single local impact for blank tread tires, rib tread tires, and lug tread tires.

Task 2. Determine the sound generated by a tire responding to single local impact.

Task 3. Deduce theoretical models for both the vibratory response and the radiated sound for single local impact.

Task 4. Establish suitable probability distributions for the succession of impacts encountered from macrotexture in asphalt and portland cement concrete surface courses common in highway construction. Relate the probability distributions to the aggregate gradation or the method of surface texturing (brushing, dragging, etc.).

Task 5. Deduce theoretical models for the noise generated by distributions of impacts in terms of the models for sound and vibration from single local impacts.

Task 6. Test the validity of the theoretical models by full scale experiments on carefully prepared lengths of test pavement.

Task 7. Test the sensitivity of model predictions to tolerances in aggregate gradations and in texturing specifications for PCC.

Task 8. Specify a low-noise pavement texture design, construct a test strip in accordance with it and demonstrate that the tire-pavement noise for various tire tread designs is as predicted, while other performance parameters such as durability measures and skid number remain at satisfactory levels.

Cost Estimate $250,000

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FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.*

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems connected with the responsibilities of the Federal Highway Administration under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by keeping the demand-capacity relationship in better balance through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements which affect the quality of the human environment. The ultimate goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge of materials properties and technology to fully utilize available naturally occurring materials, to develop extender or substitute materials for materials in short supply, and to devise procedures for converting industrial and other wastes into useful highway products. These activities are all directed toward the common goals of lowering the cost of highway construction and extending the period of maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural designs, fabrication processes, and construction techniques, to provide safe, efficient highways at reasonable cost.

6. Prototype Development and Implementation of Research

This category is concerned with developing and transferring research and technology into practice, or, as it has been commonly identified, "technology transfer."

7. Improved Technology for Highway Maintenance

Maintenance R&D objectives include the development and application of new technology to improve management, to augment the utilization of resources, and to increase operational efficiency and safety in the maintenance of highway facilities.

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* The complete 7-volume official statement of the FCP is available from the National Technical Information Service (NTIS), Springfield, Virginia 22161 (Order No. PB 242057, price $45 postpaid). Single copies of the introductory volume are obtainable without charge from Program Analysis (HRD-2), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

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